8

Shallow Foundations

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Mohammed S. Islam
California Department of Transportation

Amir M. Malek
California Department of Transportation

8.1 Introduction

The term *foundation* is often used to refer to the part of the structure that transmits the weight of and other force effects on the structure on to the ground. This is a narrow definition of foundation that can lead to problems in designing foundations for structures. A complete definition of foundation should include the soil or rock, more generally the geomaterials that provide the necessary resistances so that the structure (1) will not experience unacceptable deformation that can render it unusable for the intended purposes and (2) will remain stable or not fail to protect life and the investment made by the owners at all times during the design life. A more representative definition refers to the structure and the geomaterial components of the foundation as "structure foundation" and "foundation geomaterial" or more commonly as "foundation soils," respectively.

The main purposes of the geotechnical design of foundations for civil engineering structures are to (1) limit any deformations or movements that the foundation may experience during the design life due to the expected day-to-day average or service loads so that the structure will maintain its intended functionality and (2) maintain its stability against all kinematically admissible failure or collapse mechanisms if, for some unexpected reason, subjected to a certain load that is significantly higher in magnitude or much less likely to occur during the design life, than the expected day-to-day average load.

Structure foundations are generally grouped into two primary categories: (1) *shallow foundations* and (2) *deep foundations*. This chapter is primarily devoted to the geotechnical design of *shallow foundations*.

Basic foundation design requirements, geotechnical considerations, and the current design methodologies are presented followed by the various methods available for the analysis and evaluation of the necessary geotechnical design parameters. Both the traditional Working Stress Design (WSD) and the more recent AASHTO (2012) Load and Resistance Factor Design (LRFD) methodologies are covered. Structural design aspects of shallow foundations are also briefly addressed. Deep foundations for bridges are covered in Chapter 10.

8.1.1 Basic Foundation Design Requirements

The primary objective of any foundation design is to transfer the structure loads to the foundation geomaterials in ways such that the specified performance requirements are met in a cost-effective manner. As such, foundation design needs to consider the following basic requirements:

- The type or nature, usage, design life, and performance requirements of the structures to be supported.
- The nature, magnitude, and the likelihood of the occurrence of the various types of loadings or demand on the foundation and those of the available resistances or capacities. The loading for design need to include the following:

  The average magnitude of the loads normally expected to occur during the day-to-day operation of the structure. In WSD this load is generally termed simply as the "design load." In LRFD it is often termed somewhat misleadingly as the "unfactored load," whereas the term "design service load" is more desirable and recommended for use.

  A certain significantly higher magnitude of the normally expected day-to-day loading that has some but much lower probability of being exceeded during the design life than the design service load. In LRFD this is termed the "design factored load."

  Other rare but significantly high magnitude loadings such as those due to ground motion generated by moderate to large magnitude earthquakes at the sites located near or within some moderate distances, most commonly on the order of 100 km or less, from the earthquake source, or the sites that may experience a rare but significant magnitude storm or hurricane. The design load(s) for these events, termed in LRFD as the "extreme events," depends on many factors. A more detailed description is outside the scope of this chapter, but noteworthy to state that the design loads for these events depend to a large extent on the cost of design and construction to reduce the associated risks and the acceptable level of risks. Most often it is not likely or even feasible to completely eliminate these rare risks, in particular against those due to significant deformations during large magnitude events. For seismic design these loads are often referred to as "design seismic loads," which could be for the functional-level or the safety-level design.
- Any potential effects on the soil stiffness and strength or resistances due to construction as well as the type or nature and the magnitude of the design loadings, such as the reduction in the strength of some soils during seismic events, need to be evaluated and considered in appropriate

combinations and in conjunction with the applicable loading types, magnitudes, and their combinations. Any uncertainties in the estimated soil ultimate strength or resistances need to be considered, most commonly by using reduced strength parameters or ultimate resistances. In addition to professional judgments, the magnitude of such reduction is generally achieved in the WSD by using the concept of a factor of safety (FS) and in LRFD by using a "resistance factor (ϕ)."

Structure foundation needs to be located and designed such that environmental and geologic factors or conditions, including frost, scour, erosion, corrosion, seepage, piping of foundation soils, or future planned and anticipated developments and other human actives will not jeopardize its stability as well as functionality during the design life.

The potential effects of construction on adjacent existing structures, if any, need to be considered, and when necessary, mitigation measures are implemented to eliminate or reduce any potential negative effects to within acceptable limits. The potential of effects of any nearby planned future development on the foundation also need considerations during design.

Foundation movements or deformations due to the design service loads need to be limited to small specified amount for the structure to be useful in an uninterrupted manner or serve its intended purpose throughout the design life without needing to spend excessive amount of additional resources for repair or maintenance. These maximum limiting foundation deformations are variably referred to in the literature as allowable, tolerable, or permissible limits. These specified maximum deformation limits, mostly based on past observed or measured performances of similar structures and professional judgments, often vary widely and is a matter of significant uncertainty and long-standing confusion among many professionals. For important structures projects, the permissible deformation limits should be set based on project-specific requirements.

Foundations need to be designed to remain stable against all possible types of failure or collapse mechanisms with an acceptable level of reliability when subjected to the design factored loads and, where applicable, the design loads due to extreme events. The acceptable degree of reliability against all potential instability mechanisms, including the magnitude of the design factored or seismic design loads discussed above, is represented by FS in the traditional WSD and by a unique combinations of load and resistance factors in the LRFD methodology, for each of the potential instability or failure mechanisms.

Additional discussion on the various aspects of geotechnical or soil deformations and stability aspects of foundation design is presented in the following sections for both the WSD and the LRFD methodologies. FHWA (2002) provides guidelines for the determination of soil and rock properties for use in the analysis and design of bridge foundations.

8.1.2 Basic Geotechnical Considerations

Foundation geomaterials are generally classified as soil or rock. Both soil and rock, even at a given site, can and often do vary, sometimes significantly, in composition and engineering characteristics. Yet, often for simplicity in foundation design, soils are restrictively classified into only two groups: *cohesive soils* and *cohesionless* soils. Often, it is difficult and requires careful considerations to make a distinction between these two types of soils in real applications. For many applications, an inaccurate distinction can lead to trouble. Furthermore, foundation soil profiles commonly consist of layers of significantly different soil types or characteristics, rather than the uniform and homogeneous assumed in many in the examples in the textbooks or by some practicing engineer for many projects. Some level of simplification is essential for most project sites; however, oversimplification or inaccurate representation of the foundation soil conditions can be problematic.

The rate of the load application and the permeability of the foundation geomaterials need to be considered in the foundation design, especially when classifying a certain soil as either cohesive or cohesionless. For the purpose of foundation design, it is more appropriate and useful to classify foundation soils as either *coarse-grained* or *fine-grained* compared to the often used "cohesionless" or "cohesive," respectively.

The later classification, if used without a sound understanding of how soils behave when loads are applied at different rates, can be problematic. Additionally, in many parts of the world, for example, the Western United States, often a clear distinction cannot be made between soils and what geologists classify as bedrock or formation material from the geotechnical engineering points of view. These later geomaterials, in many engineering aspects, fall between soil and what engineers normally understand as rock—the relatively strong earth materials with distinct geologic features. For geotechnical engineering purposes, these very dense or very hard soil-like geologically classified bedrock materials have been termed as "intermediate geomaterial" (O'Neill et al. 1996) or IGM. Similar to soils, IGM has been classified into two categories: cohesive IGM and cohesionless IGM.

Soil behavior under applied loads is significantly affected by the presence of water in the pores or voids, and how quickly relative to the rate of the load application the pore water can flow out of the stressed foundation soil zone. Soil, when subjected to loads, tends to change in volume. For this volume change to occur in saturated soils, the pore water that is considered relatively incompressible compared to the soil skeleton needs to flow out of the stressed soil zone.

The rate at which the pore water can flow out depends on the soil permeability, the length of the flow path, and other boundary conditions.

The generation of excess pore pressure and its potential effects on the soil strength also depends on the rate of loading. At the rates in which the static permanent loads are generally applied, little or no excess pore pressures are usually generated in clean coarse-grained soils. In these cases, drainage of the pore water and the resulting volume change occur instantaneously compared to the rates of loadings, including those generally expected of for the day-to-day or service live or transient and temporary loads. For this reason, such soils are often identified as "free-draining." However, this categorization, unless done carefully, could be misleading because even these so-called free-draining soils may experience significant excess pore water pressure when the rate of load application is very fast, such as that occurs during earthquake events.

On the other hand, most fine-grained soils experience initial excess pore on the same order of magnitude as the applied stress even when static permanent loads are applied at the rates normally expected of. Whereas, when the loads are applied at a faster rate such as those expected of for the day-to-day live loads and for the seismic loads, little or no excess pore pressure generates in most low-permeable fine-grained soils.

The change in the pore water pressure above that existed for the equilibrium conditions (u_o) before the application of the load is termed as the "excess pore pressure" (Δu). It can be positive (increase) or negative (decrease) depending on the initial "state" of the soils. The "state" of a soil element is expressed in terms of its initial void ratio and the average effective confining stress. The initial state of a soil element implicitly incorporates its past stress history—which is an important factor in this regard.

Shear strength of soils depends on the effective stress. Depending on the permeability the soil effective stress, and thus its shear strength, can vary over the period of time starting from the initial load application and ending with the complete dissipation of the generated excess pore water pressures. As stated above, the generated excess pore water pressure can be either positive or negative resulting in a short-term decrease or increase in the soil strength, respectively. For most projects the dissipation of the excess pore water pressure can be expected to complete well before the end of the design life. For soils experiencing positive excess pore pressure, the initial strength, that is when the excess pore pressure is the maximum, will govern the stability conditions. On the other hand, for soils experiencing negative excess pore pressure, the long-term strength, that is when the excess pore pressure is fully dissipated,

will govern the stability conditions. Often it is difficult to determine whether the generated excess pore pressure will be positive or negative at a given project site. For such cases, analyses should be performed to evaluate both the (1) short-term or initial stability conditions when the loads are applied and (2) long-term or the stability conditions that may exist once the dissipation of any generated excess is completed.

Additional and sometime more significant difficulties are associated with the determination of the magnitude of the excess pore water pressure that may be generated at a project site. For most projects, even for static design, these difficulties are avoided by using the total stress based or undrained strength for the fine-grained soil layers in the evaluation of the short-term or initial stability conditions that may exist during or for a short time after the application of the loads.

The long-term or the stability conditions that may exist once the dissipation of the excess pore water, if any were generated, is complete and is conveniently evaluated based on the effective stress-based strength for all types of soil layers.

For seismic design, stability conditions during or immediately following the design ground motion event is generally evaluated by the utilizing appropriate fully undrained strengths for the nonliquefiable fine-grained soils layers.

When complete liquefaction of a soil layer is predicted at any instant of time at which stability conditions are to be evaluated, the undrained residual or steady-state shear strength (S_r) for liquefied soils recommended by several researchers (Seed and Harder 1990; Stark and Mesri 1992; Olsen and Stark (2008); Idriss and Boulanger 2008; Robertson 2013) is generally used in lieu of the effective stress-based strengths.

For all other soil layers, the potential effects of the development of partial excess pore water pressures, if any is predicted to develop, at the instant of time during or immediately after the end of the design ground motion event at which stability conditions are evaluated should be considered in the seismic stability analyses.

The terms "cohesionless' and "cohesive" are sometime mistakenly used as representing soils whose shear strengths are characterized with friction and cohesion only, respectively. This could lead to a problem in the foundation design. In the long term following the application of all types of permanent loads, most soils are likely to behave as purely frictional materials, and inclusion of any cohesion in the long-term stability evaluation can lead to dangerous consequences.

8.1.3 Definitions

A shallow foundation, often referred to as a spread foundation or simply as footing, may be defined as the one with an embedment depth (D) below the lowest adjacent ground on the order of its effective width (B'), as illustrated in Figure 8.1. The D/B' ratio for shallow foundation commonly ranges from 0.25 to 1.0 but may be as high as 2.5 (Munfakh et al. 2001).

8.1.4 Types of Shallow Foundations

Commonly used types of shallow foundations include individual footings, strip or continuous footings, combined footings, and mat or raft. Shallow foundations or spread footings derive their vertical load–supporting capacity entirely from their base bearing resistance, whereas deep foundations derive their axial load–supporting capacity from both side and base resistances.

Shallow foundations are generally more economical to design and construct. Therefore, shallow foundations are often preferred over deep foundation if the geomaterials at shallow depths are firm and suitable for supporting the anticipated structure loads. Even in firm ground, use of shallow foundations may not be feasible due to other considerations such as the unusually high loads, vertical or lateral or both, difficult underwater construction conditions, and limited right-of-way.

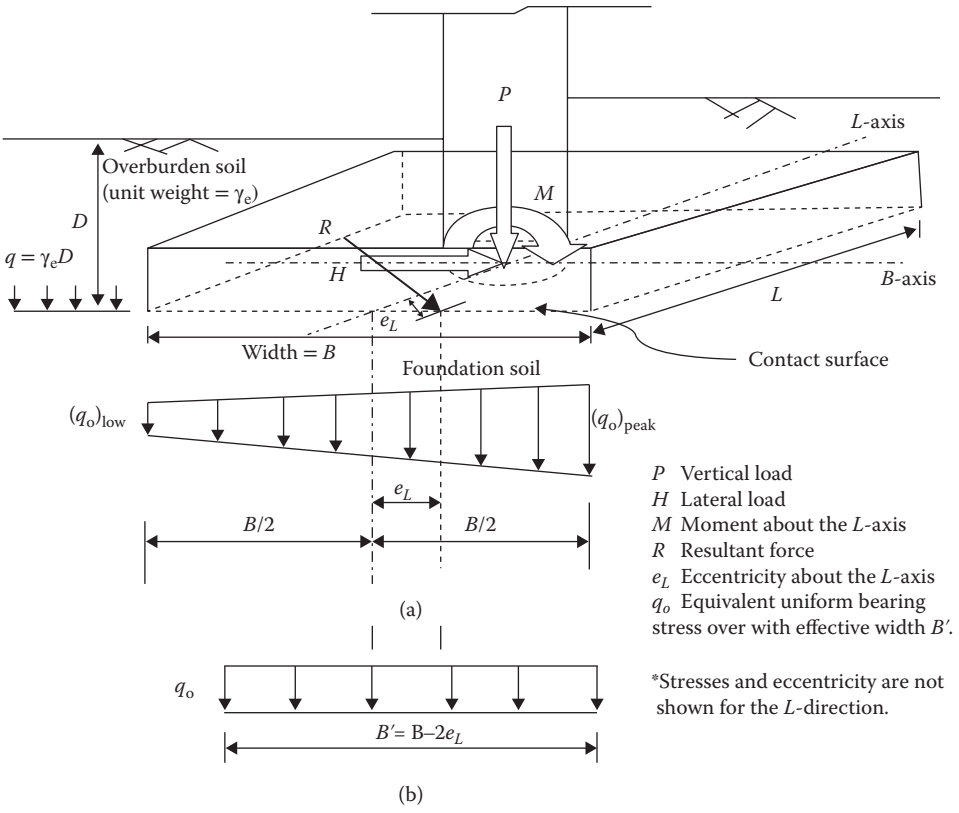


FIGURE 8.1 Definition sketch for shallow foundations. (a) Idealized contact stress distribution and (b) stress distribution for geotechnical analysis and design.

8.2 Design Methodologies

In the WSD, performance requirements for the design are specified based on past experiences and, to a great extent, on professional judgments. Stability requirements are generally specified in terms of an FS defined as the ratio of the available ultimate capacity to the expected day-to-day average destabilizing load or demand during the design life. However, both the parameters used to define the FS are subject to different degrees of uncertainties. The concept of the FS used in the stability design of a structure and, in some cases, components thereof against the various potential modes of failures has been used to incorporate these uncertainties in an empirical manner. This design concept, however, does not permit the evaluation of the uncertainties and the resulting reliability of the design in a systematic, uniform, or universal manner.

As a simple example, consider a case where spread footing foundations for two adjacent bridge structures are similar in all aspects, except they are located at two different nearby locations with different foundation soil types and conditions. One of the bridges also has a fewer number of supports than the other. Footings for both the bridge structures were designed for an FS of 3.0 against soil bearing capacity–type failures. Although the spread footings for each structure were designed for the same FS, the reliabilities associated against bearing capacity failures are not likely to be the same, because both the loads and the soil bearing capacities for each structure are likely to be subjected to different degrees of uncertainty. Furthermore, the estimated magnitude or the degree or even the likely range of the uncertainty or reliability associated with the bearing capacity failure for neither of these bridges is known.

In the aforementioned example, it would have been desirable and prudent to design such that the same degree of reliability against bearing failure is provided for each structure and also that the

magnitude of this reliability is known. These would have permitted to optimize these designs and meet the performance requirements with a known and the same level of reliability. Furthermore, if needed, the specified performance requirements could have been evaluated in terms of their effects on the cost and other aspects of the design and modified or adjusted within the limits of the overall project requirements and the acceptable level of reliability.

The object of the LRFD methodology is to address some of the limitations, including those discussed earlier, associated with the WSD methodology by systematic evaluation of the uncertainties involved in the determination of both the demand or load and resistance or capacity and incorporating those uncertainties in the design to achieve an acceptable and preselected level of reliability against a given mode of instability. It achieves these objectives by utilizing the concepts of the *load factors* (γ_i) and the *resistance factors* (ϕ_j).

LRFD also permits to specify for and design to achieve or provide the same degree of reliability for similar structures, in terms of use and/or importance irrespective of their locations and other differences. LRFD also provides for clear identification of the various performance and stability or failure conditions or mechanisms (states) and their limiting magnitudes (limits) for which to perform analysis and design a given structure for so that it will perform during the entire design life in the manners intended by the owners as well as the designers.

In summary, both WSD and LRFD involve analysis and design to limit foundation movements due to the service or day-to-day average load to maintain serviceability of the structure and to ensure adequate stability or safety against all applicable instability or failure mechanisms. Thus, the geotechnical deformation and soil strength parameters necessary for the evaluation of the various stability conditions are generally applicable to both WSD and LRFD except that, as noted where necessary, there are differences in some terminologies and techniques by which the stabilities are evaluated and minimum requirements are met by the design.

The mechanisms involved in the development of foundation deformations or movements, generally referred to as *soil–foundation interactions* (SFIs), are often complex. For practical purposes or design, most often it is adequate to decouple the foundation movements due to soil flexibility from those due to the structural flexibility of the footing and/or the supported structure or parts thereof. This permits the geotechnical and structural deformation analyzes to be performed separately and more easily by the respective designers.

For small deformations, that is, for serviceability design, this approach should be sufficiently accurate for most projects. Owing to soil nonlinearity and the complex interactions between the foundation soils and the structure foundation(s), additional considerations are required when soil movements or deformations are large, especially when structure foundations are subjected to kinematic forces. In such cases, application of the simple decoupling methods is not likely to provide sufficiently reliable results, in particular for important structures. For important structures or complex situations, a detailed SFI or a soil–foundation–structure interaction (SFSI) analyses are usually required to estimate and be able to evaluate foundation movements more accurately.

For the serviceability design of spread footing foundations, geotechnical consideration is limited to the movements of the footing due to the soil flexibility or deformations and/or rigid movements of the soil–foundation system. These types of footing movements are generally referred to as the *geotechnical deformation or movements*. Deformations of the footing and the supported structure, or the elements thereof, due to the structural flexibility of the footing due to applied loads, including those due to soil pressures and kinematic movements of the soils, if any, are generally referred to as *structural deformation*. Discussion in this chapter is limited only to the geotechnical movements of shallow foundations.

Similarly, in this chapter stability considerations of shallow foundations are limited to those associated with the soil strength. These types of foundation stabilities are referred to as *geotechnical stabilities*.

Structural deformations and structural stability conditions are not covered in this section. For design, however, geotechnical movements and structural deformations need to be combined to evaluate the estimated total deformations or movements of the foundation and the supported structure or its parts thereof.

All applicable geotechnical and structural stability conditions or failure mechanisms must be identified, evaluated, and the footing designed to ensure adequate stability or an acceptable level of reliability against failure or collapse of the foundation.

8.2.1 Working Stress Design

The WSD and the allowable stress design (ASD) methods are similar, and for shallow foundation design involves (1) evaluation of and designing to limit movements to meet serviceability needs and (2) evaluation of and designing to achieve adequate stability against all kinematically admissible collapse or failure mechanisms. In WSD, all stability conditions are evaluated, and their adequacy verified by utilizing the concept of FS. For any given instability or failure mechanism, FS is defined as the ratio of the available total ultimate resistance or capacity to the total destabilizing load or demand.

In some WSD analyses, such as those performed to evaluate soil mass or global slope stability including the spread footing foundations, the FS is sometimes defined as the ratio of the unit shear strength (τ_f) or the corresponding shear strength parameters, such as the cohesion (c_f) and the friction angle (φ_f) of the soils to the actually mobilized magnitude of the corresponding strength parameters. For example, when the short-term global stability of slope consisting of fine-grained soils and containing a shallow foundation is to be evaluated using the total stress strength, that is the undrained shear strength (S_u), the FS safety may be defined as equal to (S_u/S_d), where S_d is the fraction of the total available undrained shear strength that is estimated to have been mobilized to support the destabilizing loads.

8.2.1.1 Foundation Movements or Deformations

For the serviceability design of shallow foundations, geotechnical movement or deformation include the evaluation and design for the following soil–foundation response parameters when the footing is subjected to the design service load.

- Settlement or uplift movements
- Lateral movements or deformations
- Rotation

Lateral displacements of the supported structure may occur due to lateral loads or rotation of the shallow foundation or both and are likely to vary with the height of the structure. It is, therefore, often necessary to identify the elevation or location along the height of the structure at which lateral movement(s) is being considered.

Deformation evaluation can be difficult due to the complex SFSIs. Often it is sufficient to decouple the soil-related deformation from structure deformations and use simplified estimation methods. Discussion in this chapter is limited to the soil-related movements or deformations.

When shallow foundations are constructed on, within, or near slopes or recently placed fill embankments and adjacent to another structure, potential exists for additional settlement or deformation occurring beyond the directly stressed or foundation zones or due to loads other than those imposed by the foundation under consideration. For example, settlement of the soils below the foundation level due to the additional loads imposed by recently placed embankment fill at the abutments will add to the foundation settlement due to applied structure service loads. Furthermore, a certain type of foundations soils, known as collapsible soils, may experience significant compression due to the introduction of additional moisture.

Shallow foundation design for bridge structures is commonly concerned only with the settlement. Settlement refers to the downward movement of the foundation which occurs due to compression of the foundation soils and other reasons discussed in the preceding paragraph.

For a given structure, the maximum amount of total settlement that a footing may be permitted to experience under service load without adversely affecting the serviceability or function of the supported

structure(s) is commonly referred to as the *allowable, tolerable, or permissible settlement* (s_{perm}). The net uniform contact or bearing stress that is estimated to cause this amount of settlement is termed as the "allowable contact or bearing stress" $(q_a)_s$ in WSD and net permissible contact or bearing stress (q_{pn}) in LRFD.

The limiting settlement depends on many factors including the types and functions of the structure and the spacing between the two adjacent supports. Most often, however, the differential settlement (Δ) between the two adjacent supports is more critical than the total settlement of the foundations. Additionally, as pointed out by Skempton and MacDonald (1956) the aspect of foundation settlement that relates more directly to the potential bridge superstructure cracking is the angular distortion (β) defined as the ratio of the differential settlement (Δ) and the span length (L) between two adjacent supports.

However, large total settlement at abutment foundations with respect to the approach roadway embankment can severely impact the functionality of the bridge. Total foundation settlement at both abutments and interior support locations can also cause distress to utilities carried by the bridge structure. It should be noted that the differential settlement between two adjacent supports is of most concern when it is occurring after the supported structure is connected and achieved sufficient rigidity (i.e., postdeck construction) such that it will experience distortion if any additional differential movements of the supports occurs. In addition, the total settlement is of concern once the approach road grades and appurtenant facilities adjacent to, supported by, or connected to the bridge supports are constructed and can be affected by any additional foundation settlements.

Owing to significant variations in the types, geometry, structural details, serviceability, and other aspects of bridge structures, establishing limits for either the total support settlement or the differential settlement between adjacent supports that can be used for most, if not all, cases is challenging. Guidance in this regard is very limited, often incomplete, and mostly commonly not practical or applicable to specific bridge design project in hand. Moulton et al. (1985) and Gifford et al. (1987) provide information on the tolerable movements for highway bridges.

Based on the *AASHTO LRFD Bridge Design Specifications* (2012), angular distortion (Δ/L) between the two adjacent supports of a simply supported bridge span should be limited to 0.008 radians. The corresponding limiting angular distortion for continuous span is 0.004 radians or about half of that for the simply supported span. It should be noted that, for a given limiting angular distortion, the corresponding limiting differential settlement will depend on the span length. Therefore, project-specific evaluation and specifications for the limiting total as well as differential settlement are usually necessary. Most often professional judgment plays a necessary and important role in establishing the values of the settlement limits. More fundamentally, limiting strains that would cause unacceptable cracking in structural elements due to differential settlement of adjacent foundations can be evaluated to establish tolerable differential settlement limits.

If the estimated differential settlement between two adjacent supports founded on shallow foundation is excessive, the foundation type should be changed to deep foundation unless ground improvement to reduce foundation settlement is a viable option.

The above discussion on foundation settlement is equally applicable to upward movement of foundations. Upward movement of foundations can be due to heave of the foundation soil or uplift of the foundation. Heave of shallow foundations occurs due to increase in the volume of *expansive soils* when additional moisture is introduced into these soils. Heave usually is only a concern for lightly loaded foundations and not for most bridges, because their foundations are generally heavily loaded. Uplift movements of foundations occur due to the externally applied upward (or tensile) load on foundations. Bridge foundations, in particular shallow foundations, are not usually allowed to be subjected to service level or sustained uplift forces. Thus, uplift movement is generally not a design concern for shallow foundation used to support bridge structures.

Large differential settlement or upward movement between two adjacent supports can result in the significant redistribution of the support loads leading to additional differential settlement, which can

ultimately lead to foundation bearing failure. Excessive foundation differential settlement can also overstress or even cause failure of the supported structure or its elements. Therefore, limiting differential settlement is important not only to maintain serviceability but also to prevent collapse. Even though design requires limiting the foundation differential settlement to some relatively small magnitude, the potential effects of the estimated foundation settlement should be considered in the evaluation of loads for the stability analyses for both static and seismic design.

It should be noted that for the safety-level seismic design, foundation settlement or movements should not be a concern unless such settlement or movement is predicted to cause failure of the otherwise adequately stable structure. This can be evaluated by considering the effects of the estimated seismic settlement in the stability analysis of the structure and, when necessary, also its components.

As depicted in Figure 8.2, eccentrically loaded shallow foundation experiences rotation (α) about the central axes of the contact surface. This rotation occurs as a result of nonuniform settlement along the transverse direction due to nonuniform contact stress caused by unbalanced moment about the longitudinal central axis of the bottom surface of the foundation. This type of foundation rotation results in the titling of the supported structure. Tilt is defined as

$$\text{Tilt}(\%) = \frac{\rho}{H} \times 100 \tag{8.1}$$

where ρ is lateral displacement of the structure at height H due to the footing rotation (α). Both the foundation and the supported structure are generally assumed to be rigid, in which case tilt is constant, and the tilt angle, $i = \tan^{-1}(\rho/H)$, is equal to the angle of rotation of the footing (α).

Rotation of shallow foundation (α) due to service loads must be limited to some small value to maintain the serviceability of the supported structure. Guidance or specifications on the maximum amount of allowable or permissible rotation for spread footings or the supported structure are hard to find. It should be noted that Moulton et al. (1982) and Gifford et al. (1987) discussed earlier provide useful information on the maximum allowable limits of the longitudinal distortion or rotation that occurs due to differential settlement between two adjacent supports.

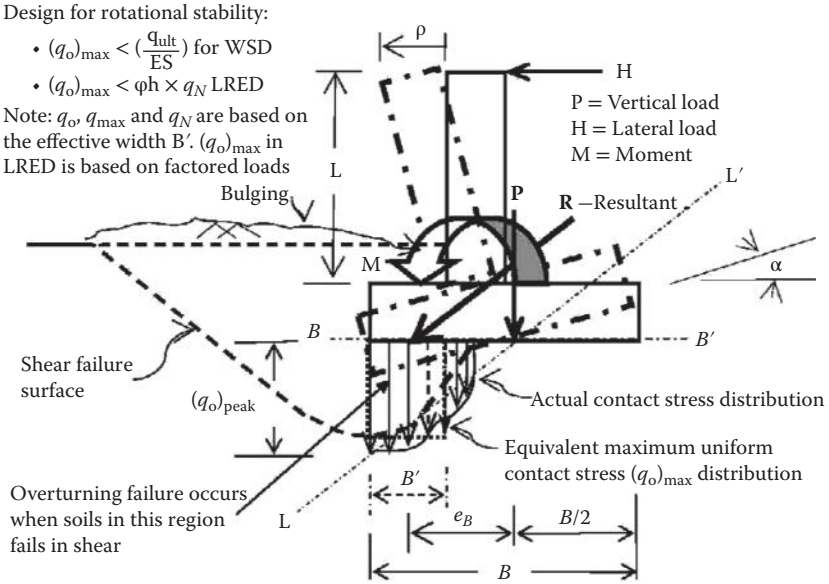


FIGURE 8.2 Rotation/tilting of shallow foundations.

The maximum amount of footing rotation that can be allowed due to service load depends on many factors, including the height and types of the supported structure, its use, esthetics, and the perception of the users. Scott and Schoustra (1968) states that as little as 1.0 in. of settlement in 20 ft. (0.5%) can be discovered, and any larger tilt is very objectionable from the esthetic and psychological perspective even if stability, both structural or geotechnical, is not an issue. TRCC (2009) specifies a limiting tilt of 1.0% for residential and other low rise buildings. This amount of allowable tilt is likely too high for most bridge structures in particular those that are relatively high. A 0.25% tilt corresponds to 0.5 and 1.5 in. of lateral displacement at the top of a 16- and 50–ft.-high column, respectively. Intermediate supports are not usually subjected to sustained lateral load. Therefore, tilting of the intermediate support columns is usually not a concern for the static design. On the other hand, abutments are commonly subjected to significant sustained lateral loads due to lateral earth pressures. A 0.25% tilt of a spread footing foundation will result in approximately 0.6 and 0.9 in. of lateral deflection at the top of a 20- and 30-ft.-high abutment walls, respectively. Therefore, depending on the height of the bridge, 0.25%–0.5% should be allowed for most bridge structures from the esthetic and perception point of view. *However, similar to differential settlement, both geotechnical and structural design must consider the potential effects of the estimated tilt due to service load in the stability analysis and design.*

It should be noted that excessive rotation of the footing due to the shear failure of the soils underneath the toe areas would result in the rotational or tilting failure, which has traditionally been termed as the "overturning," failure, of the supported structure. This aspect of footing behavior and the design requirements will be discussed in Section 8.4.

8.2.1.2 Foundation Stability or Failures

The stability conditions generally associated with shallow foundation design include the following:

- Soil bearing stability in compression
- Soil bearing stability in uplift or tension
- Lateral stability
- Rotational stability
- Hydraulic stability

Lateral stability is most often referred to as sliding stability and sometimes as lateral bearing stability. Generally, lateral stability of shallow foundations is achieved by a combination of sliding resistances at the footing base and the sides, and the soil lateral bearing, resistance, more specifically passive soil resistance, on the side of the footing toward which the resultant unbalanced lateral load is acting. Sliding resistances along the sides of the footing are not generally considered in the routine design but in some cases can be significant.

In addition to deformation, the overall or the global slopes supporting or adjacent to the spread footing foundations must be stable. The overall stability is generally analyzed utilizing the limit equilibrium methods. The spread footing foundation as well as all applicable external loads including those imposed by the structure on the foundation and the slope surfaces must be considered in the overall or global slope stability evaluation.

As discussed earlier, the WSD methodology utilized the concept of FS to ensure the stability of foundations. Design must ensure that the calculated factors of safety against all potential failure or collapse mechanisms are equal to or greater than certain specified minimum value of the FS. Typical values of the FS used in the static design of spread footing foundations are presented in Table 8.1.

For the design of spread footings for bridge structures, AASHTO (2002) specified safety factors are presented in Table 8.2.

Design requirements as well as the methods of analysis, for the so-called overturning failure of spread footing foundations have been somewhat ambiguous or inconsistent throughout the literature. As a result, this matter is given a special treatment later in this chapter.

TABLE 8.1 Typical Values of Safety Factors Used for the Static Design of Shallow Foundations

Stability or Failure Mode		Factor of Safety (FS)	Comments
Overall or slope stability		1.5–2.0	Lower values are used when uncertainties in the design are low and/or consequences of failure will not be significant. Higher values are used when uncertainties in the design are high and/or consequences of failure will be significant. Higher values are also recommended for some FS for footing founded in clay soils.
Bearing capacity		2.5–3.5	
Lateral (sliding) stability		1.5–2.0	
Overturning		1.5–2.5	
Hydraulic stability	Uplift/flotation	1.5–2.0	
	Heave	1.5–2.0	
	Piping	2.0–3.0	

Source: Data from Terzaghi, K. and Peck, R. B., *Soil Mechanics in Engineering Practice*, Second Edition, John Wiley & Sons, New York, 1967; NCHRP, Manual for the Design of Bridge Foundations, National Cooperative Highway Research Program Report No. 343, National Science Foundation, Washington, D.C., 1991; and Munfakh, G., Arman, A., Collin, J. G., Hung, J. C., and Brouillette, R. P. (2001), *Shallow Foundations*, Federal Highway Publication No. NHI-01-023, Washington, D.C., 2001.

TABLE 8.2 AASHTO (2002) Specified Factor of Safety (FS) for the Static Design of Bridge Shallow Foundations

Stability or Failure Mode		FS	Comments
Overall or slope stability		1.3–1.8	FS $\geq$ 1.3 when soil/rock parameters are based on in situ and/or laboratory testing. Otherwise, FS $\geq$ 1.5. When the footing is founded on a slope or above an earth retaining system, the corresponding FS are $\geq$1.5 and $\geq$1.8, respectively
Soil bearing stability		$\geq$3.0	Bearing capacity based on effective footing width
Lateral stability		$\geq$1.5	Based on effective footing area
Overturning		1.5–2.0	Based on summing moments about toe. FS $\geq$ 2.0 in soils and $\geq$1.5 in rocks
		$E \leq B/6$ or $B/4$	$e \leq B/6$ in soils and $e \leq B/4$ in rock
Hydraulic stability	Uplift	—	No FS values are specified. Investigation into uplift and prevention of piping are specified
	Heave	—	
	Piping	—	

8.2.2 Load and Resistance Factor Design

LRFD of shallow foundations involves identification of and analysis and design for three different groups of conditions or mechanisms called limit states, namely (1) service limit states, (2) strength limit states, and (3) extreme limit states, which include seismic design.

8.2.2.1 Service Limit States

The service limit states design pertains to the performance of the bridge with regard to serviceability during the design life of the bridge and is evaluated under expected day-to-day, operating, or service loading conditions. The various components of the expected service loads are estimated or calculated in a way similar to the traditions working stress method. Sometimes these loads are referred to as "unfactored loads." Although strictly speaking it is a misnomer, it does facilitate communication. Service ability or service limit state performance requirements are generally defined in terms of limiting or permissible deformation, primarily total and differential settlements for shallow foundations. Service limit states design also include analysis for and design to limited lateral movements and rotation, when applicable or significant.

Loads for the service limit state design of shallow foundations for bridge structures are evaluated based on the service limit state 1 load combination provided in the *AASHTO LRFD* (AASHTO 2012). As stated earlier, detailed guidance on the deformations to be used for service limit states design is not available, and the development of specifications on limit deformations for general use is complicated. Such limits among other factors depend on the owner's requirement, type of facilities or the required level of service, structure types, and distance between supports.

AASHTO LRFD Bridge Design Specifications (AASHTO 2012) provides additional information and guidance on this issue. For ordinary bridge structures, the California Department of Transportation (Caltrans 2008) limits total support settlement for simply supported bridge spans to 2.0 in. and for continuously supported bridge spans to 1.0 in. Differential settlement between two adjacent supports for continuously supported bridges is limited to 0.5 in. Additional total as well as differential settlement is allowed for ordinary bridges based on project-specific evaluation and consideration of the potential effects of larger settlements on the serviceability and stability of the structure.

Permissible deformation limits for important and unusual bridge structures should be evaluated and specified on a project-specific basis.

Furthermore, the capability of the profession at this time is limited on the evaluation of uncertainties involved with the available methods of movement or deformation analysis. As a result, reliability aspect of LRFD design cannot be implemented at this time. Thus, once the design load is available, the remainder of the LRFD service limit state design, including the types and components of deformations, and techniques and methods of analysis and design is similar to the serviceability aspects of WSD or ASD.

The other exception to the aforementioned similarities between the WSD serviceability design and LRFD service limit state design is the overall or global slope stability aspect of the shallow foundation design. Soils in slope stability analysis, similar to many other geotechnical engineering analyses in particular those related to the stability of earth and earth-supported or retaining structures, act both as load and resisting elements simultaneously. This makes the implementation of the concept of load and resistance factors, the main aspect of the LRFD, in the analysis and design. Thus, slope stability is currently evaluated using unfactored loads.

With the use of unfactored loads to determine the demand or destabilizing forces and soil capacities, reliability against sliding failure of the overall slope on the same order as that used in the WSD by utilizing an FS can be achieved in the LRFD strength and extreme limit state design by using a resistance equal to the inverse of the FS (i.e., $\phi = 1/FS$). This is possible because the loads involved with the overall stability analysis are mostly, if not all, permanent or dead loads. This is exactly what has been recommended in the current *AASHTO LRFD* (AASHTO 2012). However, currently *AASHTO LRFD Bridge Design Specifications* placed the slope stability aspect of the foundation design under the service limit state design. This has resulted in some confusion on the fundamental aspects of LRFD design methodology.

A somewhat lower resistance factor than that given should be used if the total live load relative to the total dead load involved with the overall stability analysis is significant.

Thus, it should be recognized that *AASHTO LRFD* (AASHTO 2012) includes overall or global stability under the service limit states design due to complexities involved in defining load and resistance factors when soil acts to exert both load and resistance simultaneously. Slope stability analysis for LRFD is performed in the same manner as for WSD, except that resistance factor, as stated above, is taken as the inverse of the FS determined based on the limit equilibrium methods.

8.2.2.2 Strength and Extreme Limit States

In LRFD design, both the strength and extreme event limit states pertain to the strength or stability aspects of shallow foundation. The stability and stability mechanisms considered are the same as those mentioned earlier for the WSD stability analyses.

With the respect to the design procedure, the primary difference with the WSD methodology is that in LRFD, instead of the FS and as discussed earlier, the adequate stability conditions are ensured by the combinations of load factors (γ) that generally increase the destabilizing load effects but currently

sometime, especially for earth structures and when combining different load components, also decrease the stabilizing effects of certain loads and always reduce the resistances or capacities by utilizing a co-dependent set of resistance factors (ϕ).

In LRFD, for both strength and extreme limit state designs, the factored resistance obtained by multiplying the nominal resistance by the specified resistance factor must be greater than or equal to the factored load for each mode of failure. That is,

$$\sum \varphi_i R_i \geq \sum \gamma_j Q_j \tag{8.2}$$

where ϕ = resistance factor, R = nominal resistance, γ = load factor, and Q = load.

Resistance factors specified by AASHTO (2012) for the strength limit state design are presented in Table 8.3.

AASHTO LRFD (AASHTO 2012) specifies a resistance factor (ϕ_g) of 0.67 for global stability of slopes supporting or containing a spread footing.

For LRFD extreme event limit states, the resistance factors are generally taken in the range of 0.9–1.0. For seismic design, AASHTO (2012) specifies a resistance of 0.9 for all stability or failure analysis. California Department of Transportation (Caltrans 2011a) specified a resistance factor of 1.0 for extreme event limit states design, including the safety-level seismic design of bridge structures.

It should be noted that for the safety-level seismic design, limiting deformation should not be a consideration unless the estimated deformation lead to collapse of the structure. Some agencies require a functional-level seismic design in which deformations are required to be limited so that a specified level of functionality of the bridge will be maintained after more frequent ground motion events. However, a significantly lower level of ground motion is specified for the functional-level design than that for the safety-level design.

In addition to applicable loads, changes in soil capacities or resistances and other potential effects such as ground movements inducing kinematic forces must be considered in the seismic design for safety-level design earthquake ground motion and also, when required, for functional level design earthquake ground motion. Neither the loads nor the resistances specified or applicable for one limit state should be considered in combination with a different limit states. For example, estimated effects of soil liquefaction downdrag or kinematic forces due to lateral spreading or deformation due the design seismic event must not be considered in conjunction with the service or strength limit state design.

TABLE 8.3 Resistance Factors for Strength Limit States Design of Shallow Foundations

Geotechnical Nominal Resistance	Notation	Method/Soil/Condition	Resistance Factor
Bearing resistance	ϕ_b	Theoretical method (Munfakh et al. 2001), in clay	0.50
		Theoretical method (Munfakh et al. 2001), in sand using *CPT*	0.50
		Theoretical method (Munfakh et al. 2001), in sand using *SPT*	0.45
		Semiempirical methods (Meyerhof 1956), in sand	0.45
		Footings on rock	0.45
		Plate load test	0.55
Sliding	ϕ_τ	Precast concrete placed on sand	0.90
		Cast-in-place concrete on sand	0.80
		Cast-in-place or precast concrete on clay	0.85
		Soil on soil	0.90
	ϕ_{ep}	Passive earth pressure component of sliding resistance	0.50

Source: Data from AASHTO, *AASHTO LRFD Bridge Design Specifications*, Customary U.S. Units, 2012, American Association of State Highway and Transportation Officials, Washington D.C., 2012.

Note: Modified by the authors to refer to the correct reference. Meyerhof (1957) provides theoretical methods, whereas Meyerhof (1956) provides semiempirical methods for sand using both SPT and CPT.

Resistance factors depend, among other factors, on the load factors and the methods of resistance evaluation. The "nominal resistance" for each type of stability evaluation is determined based on calibrated theoretical, semiempirical, or test methods. The definition of the ultimate capacity or the nominal resistance may vary from method to method. Thus, it is important to note that nominal resistance to be used in association with a specified resistance factor should not be modified based on arbitrarily defined deformation or settlement limits. In general, the nominal resistance should be the resistance at which the associated structure or the component thereof collapses or fails and should not be associated with any limiting deformation in the sense used in serviceability evaluation. Owing to certain inherent theoretical or testing limitations, as well as some practical considerations that prevent from defining a universally accepted definition of the failure or collapse, some nominal resistance determination methods uses certain magnitude of deformation as the limiting deformation to evaluate nominal resistance. However, such deformation limit is usually large to constitute a failure and need not be considered in design. This is because, by definition, the purpose of the stability evaluation is to prevent collapse with certain acceptable and generally degree of reliability not to limit deformation or settlement. Limiting deformation that, if any, used to define the nominal resistance in the original determination method, under the factored load that has a very low probability of occurrence, will necessitate an extremely conservative design beyond that necessary or required by the *AASHTO LRFD* specifications (AASHTO 2012).

It should be noted that load factors for a given load combination (as given in Chapters 5 and 6 of *Bridge Engineering Handbook, Second Edition: Fundamentals*) can be different for different stability or failure modes. Additionally, a load factor may have a specified single value or specified minimum and maximum depending on the type of effects (e.g., stabilizing/positive or destabilizing/negative) of the associated load component on the stability condition being evaluated.

8.3 Settlement and Bearing Stability Considerations

Figure 8.3 illustrates typical load–settlement relationships for spread footing of width B when subjected to a vertical load P resulting in a uniform contact or bearing stress of q_o. It may be noted the vertical load P is assumed to be applied concentrically. The concentric loading can be considered as a special case of eccentrically loaded footing where eccentricity $e = 0$, that is, the effective width $B' = B$. This permits the use of the effective footing width in all geotechnical analyses and designs.

Geotechnical correlations for footing settlement and bearing capacity depend on the size of the loaded width of the footing. These correlations are always developed, and presented in the literature and in the following sections, as a function of the width parameter B. For this width parameter B to be the same as the actual footing width, the load P must be applied concentrically so that the footing is uniformly loaded over the full width. For eccentrically loaded footing, the width parameters B must be replaced with the effective width $B' = B - 2e$. This presents a challenge when performing geotechnical analyses, evaluations, and design, because both the footing width and the eccentricity parameters are not known at this time. In order to obtain the most cost-effective or optimal design, an iterative design procedure involving significant interaction between the geotechnical and the structure designers would be needed

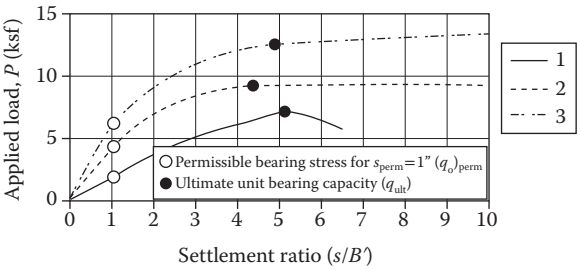


FIGURE 8.3 Load–settlement curves for spread footings.

to complete the design. In reality, this is often difficult and not executed resulting in less than optimum design. These difficulties can be easily avoided by following the procedure recommended below.

With the understanding that the concentrically loaded footing is a special case of eccentrically loaded footing with $e = 0.0$ or $B' = B$, it can be easily seen that it is appropriate, and at the same time more convenient, to simply consider the footing width parameter B in the geotechnical correlations, as presented in the current literature, and analyses and designs as the effective footing width. This can be accomplished in two different ways: (1) replace the symbol B with the symbol B' in all geotechnical correlations and analyses and (2) consider the parameter B in these correlations as representing the footing effective width (B') rather than the actual width when performing geotechnical analyses and design. For structure design, the actual footing width is used.

Based on the this, irrespective of the symbol used (i.e., B or B'), it is necessary to understand that the geotechnical correlations, and thus the design geotechnical parameters, are conveniently presented in terms of the effective footing width irrespective of the actual footing width and the load eccentricity. This permits the geotechnical designers to provide plots or tables of the recommended geotechnical design parameters for settlement and stability analyses as a function of the footing effective width. The range of the effective width for which these parameters are provided should be chosen, so that it well encompasses the effective width of the design footing. The structure designers can use these data in an iterative manner to size the design footing with the need to request updated geotechnical design parameters and recommendations every time either the footing size or the eccentricity changes during the analysis and design process until the most optimal design is obtained, which not only satisfies all applicable deformation and stability requirements, but also is the most cost-effective.

As indicated in Figure 8.3, the settlement ratio (S/B') increases as the applied load increases. If the applied vertical load is P, then the uniform unit contact or bearing stress (q_o) is obtained by

$$q_o = \frac{P}{A'} \tag{8.3}$$

where $A' = (B') \times L'$ is the effective area of the footing.

The maximum vertical load P_{max} that can be applied on the footing is the peak load (curves 1 and 2) or the load at which settlement continues to increase with little or no further increase in the load (curve 3). When these conditions are reached, the footing is considered to have failed. In other words, the applied load becomes equal to the ultimate bearing capacity of the footing (Q_{ult}). That is,

$$Q_{ult} = q_{ult} \times A' = P_{max} = (q_o)_{max} \times A' = Q_N = q_N \times A' \tag{8.4}$$

where q_{ult}, = uniform unit ultimate bearing capacity = Q_{ult}/A' for WSD; q_N = uniform unit nominal bearing resistance in compression = Q_N/A'; $(q_o)_{max}$ = equivalent uniform contact or bearing stress = P_{max}/A'; and A' = effective footing width = $B' \times L'$.

In this case as the load is applied concentrically, $A' = A = B \times L$.

As stated earlier, the "nominal resistance" in LRFD is synonymous with the "ultimate capacity" in WSD. Therefore, ultimate capacities and nominal resistances are used interchangeably in this section and applicable to both WSD and LRFD. Distinctions in other geotechnical parameters, when necessary, are clearly noted.

With reference to Figure 8.3, it should be noted that

- For a given footing size, the settlement at which the applied uniform contact stress (q_o) becomes equal to the ultimate unit bearing capacity (q_{ult}) of the footing, as defined earlier, depends on the types and conditions of the foundation soils and the rate of loading.
- The total settlement at which the applied uniform contact stress (q_o) becomes equal to the ultimate unit bearing capacity (q_{ult}) depends on the effective width (B'). The larger the footing the greater is the settlement at which a footing reaches one of its failure states, as shown in Figure 8.4. If one

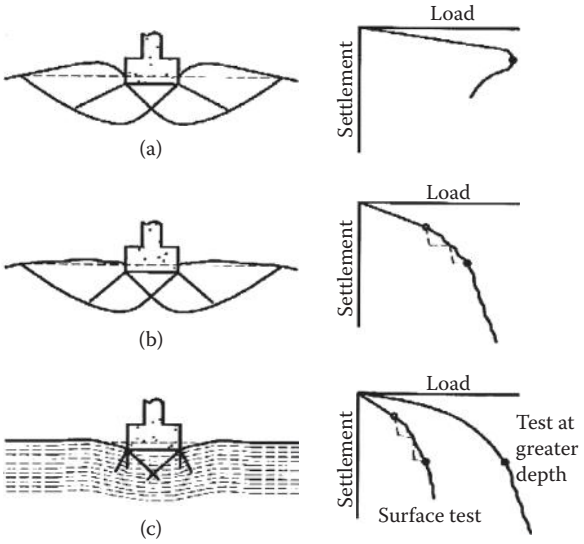


FIGURE 8.4 Bearing capacity failure modes for shallow foundations. (a) General shear; (b) local shear; and (c) punching shear. (Adapted from Vesic, A. S., *Foundation Engineering Handbook*, Edited by Winterkorn, H. F. and Fang, H. Y., Van Nostrand Reinhold Company, 1975.)

of these states is reached, the footing is said to have failed in bearing capacity. For all practical purposes, the corresponding settlement is very large and is not a consideration in that required safety against reaching such a state is provided by the FS in WSD and by the combination of load and resistance factors in LRFD strength and extreme limit states.

Model (Vesic 1963) and centrifuge (Kutter et al. 1988) testing has shown that settlement of spread footing foundation ranges from approximately 5% B' in dense soil to 20% B' in loose soils. In practice, an average settlement of approximately 10% B' is often considered at the failure settlement for spread footings as well as base failures of deep foundations (Randolph 2003). Therefore, it is clear that the total settlement at which spread footings, except those with very small width, fail is generally very large compared to the settlement that can be permitted to occur to maintain the serviceability of the supported structures, including bridges.

It should however be noted that the bearing capacity estimated based on the theoretical methods discussed later corresponds to this large displacement. The generally used FS in WSD and the load and resistance factors for stability analyses are based on the ultimate capacities. As such, the use of any other arbitrary definition of bearing capacity, such as that corresponding to a relatively small magnitude of the footing or support settlement (such as 0.5 or 1.0 in.), is inappropriate for use with the currently recommended factors of safety and/or the load and resistance factors for foundation design. This misleading concept is probably the result of misunderstanding on the use of such definitions in the past for recommending and use of the so-called presumptive bearing capacity that was developed based on limiting settlement to some small magnitude such as 0.5 or 1.0 in. (Scott and Schoustra 1968; Lambe and Whiteman 1979; Terzaghi et al. 1996). The "presumptive bearing capacity" is in fact a misnomer, which should be referred as the "presumptive allowable or permissible bearing stress" due to design load in WSD and the design service (or unfactored load) in LRFD to be accurate.

As discussed earlier, to achieve an acceptable degree of reliability or confidence in the design against shear failure, the bearing capacity of the footing that can be used in the WSD design reduces to some lower value by dividing by the FS. The corresponding reduced unit bearing capacity is termed as the "allowable unit bearing capacity" $(q_a)_c$ in WSD. The footing is sized so that the applied maximum

uniform contact or bearing stress $(q_o)_{max}$ due to service load is equal to or less than the $(q_a)_c$. That is, for WSD,

$$(q_o)_{max} \le (q_a)_c = \frac{q_{ult}}{FS} \tag{8.5}$$

In LRFD the nominal bearing resistance used in the design is reduced by multiplying by the resistance factor (ϕ_b). This reduced bearing capacity is termed as the "factored nominal unit bearing resistance" (q_R). This footing is designed such that the factored maximum uniform contact or bearing stress, which can be designed as $(q_o)_{max-f}$ for the strength as well as the extreme limit states, is equal to or less than the reduced or factored unit bearing capacity (q_R). That is, for LRFD,

$$(q_o)_{max-f}\left(\frac{P_f}{A'_f}\right) \le q_R = \varphi_b q_N \tag{8.6}$$

It should be noted that $(q_o)_{max}$ in Equation 8.5 is based on the working or design load used in WSD. In Equation 8.6, the applied maximum equivalent contact stress $(q_o)_{max\text{-}f}$ is based on the design factored load (P_f) and A'_f is the corresponding effective footing area for the LRFD strength limit state.

It is important to recognize that both the permissible total and the differential settlement for structures should not depend on the footing size, rather on some factors such as the span length, esthetics, and so on, discussed earlier that are related to the type and other characteristics, and use of the supported structure. Thus, for a given structure, the maximum amount of both total and differential settlement need to be defined in terms of some limiting values that should not depend on the size of the footing or even the foundation type.

It is also clear from above that if the footing sizes at the different supports of a given structure are different, $(q_a)_c$, and thus $(q_o)_{max}$, used on WSD will be different for different footings even for the same foundation soil conditions. In this case if the footings at each support are loaded up to the corresponding $(q_o)_{max}$, the corresponding settlement at each support is most likely to be different. Furthermore, contact or bearing stress–settlement curves are highly nonlinear and their shape depends on the types and conditions of the foundation soils. As a result it is not possible to design spread footings based only on $(q_a)_c$ in order to be able to limit the maximum total settlement under the service load to the specified permissible limit (s_{perm}), which should be the same for all the supports to reduce the risk of unacceptable different settlements between supports.

Therefore, in both WSD and LRFD service limit state design considerations must be given to limit the total settlement to the same predetermined value or the permissible settlement limit (s_{perm}) at each support irrespective of the footing size and the foundation soil conditions. This is achieved by first evaluating the maximum equivalent uniform contact stress $(q_a)_s$ or q_{perm} that can be applied on the footing without exceeding the specified permissible settlement limit (s_{perm}), as shown in Figure 8.5. Thus, for WSD,

$$(q_o)_{max} \le (q_a)_s \tag{8.7}$$

For LRFD service limit state,

$$(q_o)_{max} \le q_{perm} = \frac{P_s}{A'_s} \tag{8.8}$$

where P_s is the total net design vertical load on the footing and A'_s is the corresponding effective footing area for LRFD service limit state.

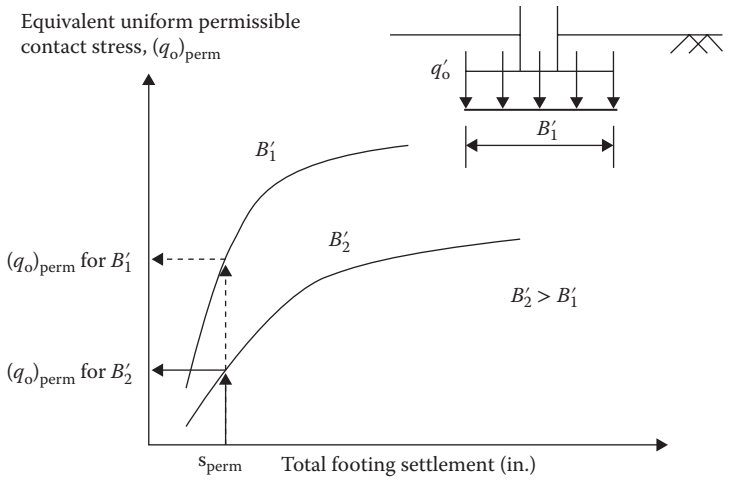


FIGURE 8.5 Plots of equivalent uniform permissible contact stress versus the footing effective width (B').

Note that unlike LRFD where the design loads for the standard static design (i.e., service and strength limit states) are different, there is only one value for the design load in WSD. Thus, the corresponding applied maximum equivalent uniform contact stress $(q_o)_{max}$ must be less than both $(q_a)_c$ and $(q_a)_s$. In other words, the footing must be designed such that $(q_o)_{max}$ is less than or equal to the lower of $(q_a)_c$ and $(q_a)_s$. Traditionally, the lower of the two parameters $(q_a)_c$ and $(q_a)_s$ has been misleadingly termed as the "footing allowable bearing capacity." This has often been the cause of confusion. Herein, the lower of these two parameters is termed as the "allowable contact or bearing stress" (q_a). That is

$$q_a = \text{lower of } \left(q_a\right)_c \text{ and } \left(q_a\right)_s \tag{8.9}$$

For WSD, the footing design requirements to meet both the safety by an FS (i.e., Equation 8.5) and the serviceability requirement as specified by a limiting settlement s_{perm} (i.e., Equation 8.7), can be replaced with the following requirement:

$$(q_o)_{max} \leq q_a \tag{8.10}$$

It should be noted that if the footing is designed by satisfying the above either WSD or LRFD bearing stability and the limiting settlement requirements, it is neither necessary nor appropriate to consider the amount of settlement, however high it may be, at which shear failure of the footing occurs. Only exception would be if the footing unit ultimate bearing capacity (q_{ult}) or the nominal unit bearing resistance (q_N) of the footing is defined based on the settlement criteria and the corresponding FS and resistance factor (ϕ_b) are specified by considering the associated definition of the footing capacity. At this time, this not a common practice for the design of spread footing foundations.

For the same reason, the deformations at which the ultimate capacity or nominal resistance against any of the other modes of failure or stability conditions are not a design consideration. Such deformations constitute failure, and the requited safeguard is provided by the FS in WSD and by the load and resistance factors in LRFD.

In determining the footing bearing capacity in this example, we can include in the ultimate uniform contact stress $(q_o)_{max}$ the loads due to the structure component of the footing and the soils above the footing area of the footing with the structure load. In this case, the term "gross" is used as an adjective to the bearing capacities discussed earlier. If the $(q_o)_{max}$ were due to the structure load only, the unit bearing capacity (q_{ult}) evaluated above would termed as the "net" ultimate unit bearing capacity. For ease of

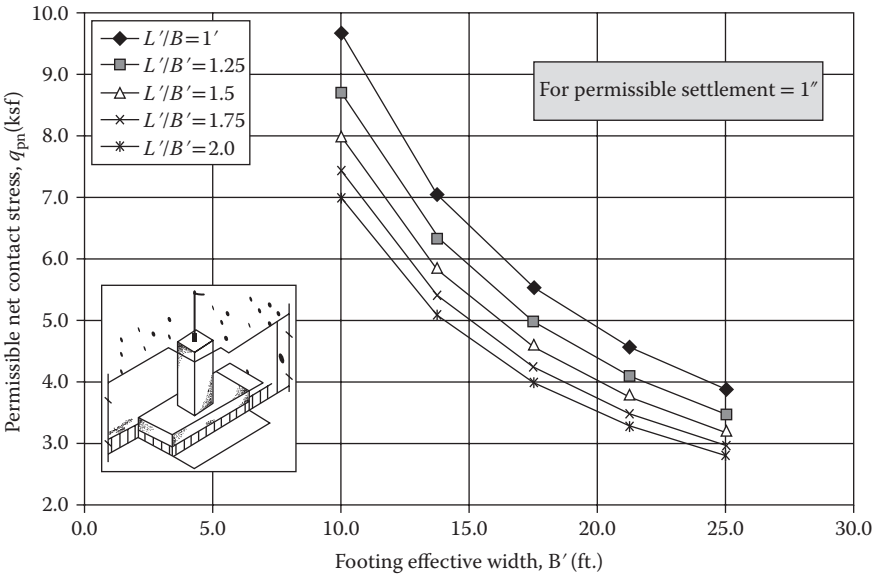


FIGURE 8.6 Schematic plots of permissible equivalent uniform contact stress versus footing effective width (B').

calculations and design, and sometimes due to small differences between the two conditions, the gross bearing capacities are generally in the analysis and design for spread foundations for bridge structures.

It should be noted that the soil settlement depends on the net stress increases. Thus, unlike bearing stability, net bearing stresses should be used in the settlement analysis. However, often when footings are founded on relatively shallow depth, differences settlement estimated by using net and gross applied bearing stresses may be small.

In general, shallow foundations design should start with establishing the appropriate embedment depth of the footing (D) and making a rough estimation of the likely range of the footing size or the width.

In LRFD, the next step is to develop plots of the the permissible equivalent uniform contact stress (qo) perm and equivalent uniform factored nominal bearing resistance (qR) as a function of the footing effective width (B), as shown in Figures 8.6 and 8.7, respectively. Similar plots can be developed for the WSD.

For rectangular footing, these plots should be generated for a range of the footing effective length to the effective width ratio L'/B'. In addition to the embedment depth D, information necessary to develop these plots include the permissible settlement limit (s_{perm}), the resistance factor (ϕ_b), and the foundation soil profile with appropriate design soil parameters.

The range of the effective with (B') can be selected based on preliminary site conditions and the loads. It should be noted that the footing width used in the geotechnical analysis and design calculations is the effective width (B'). Thus, unless noted otherwise, the footing width (B) throughout this chapter refers to the effective width (B'). However, in structural analysis and design of the footing, the actual width (B) is used. For concentrically loaded footings, the effective width is the same as the total width.

8.4 Rotational Stability

Over the years, the analysis methodologies and design requirements for rotational stability spread footings, resulting in the tilting failure of the supported structure have been inconsistent and major sources of confusion. Tschebotarioff (1970) presents several cases of bridge abutment failures due to rotational failure of spread footing foundations. Bowles (1982) reported that five different methods are available in the literature for evaluating rotations, none of them were in good agreement with one another. Earlier textbooks on foundation design (e.g., Tschebotarioff 1951; Karol 1960) analyzed such stabilities

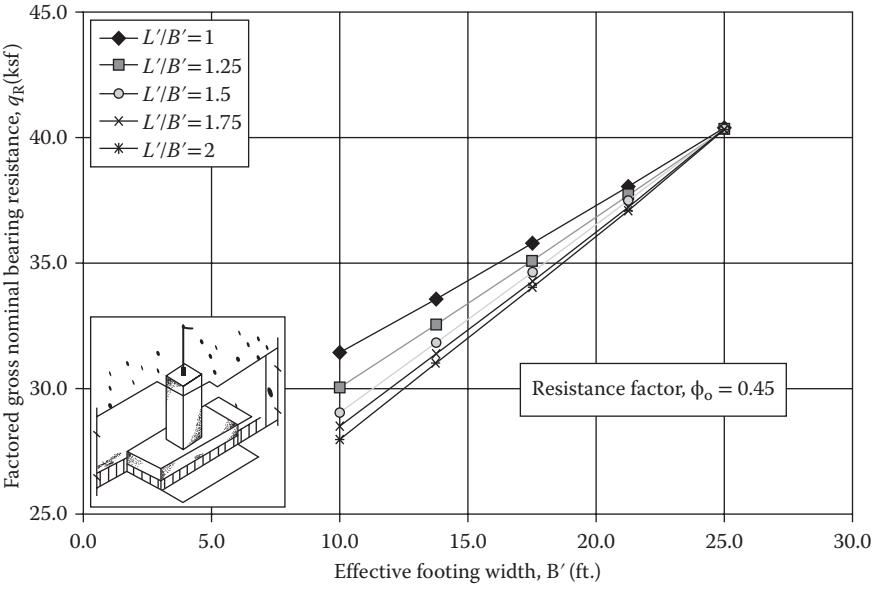


FIGURE 8.7 Schematic plots of equivalent uniform factored gross nominal bearing resistance versus footing effective width (B').

as overturning stability. Stability was achieved by using an FS against overturning defined as the ratio of moment due to the destabilizing forces to that due to the resisting forces about the toe of the footing.

Many later literatures (e.g., NAVFAC 1986, Bowles 1988, AASHTO 2002) included a limiting eccentricity requirement as an additional design requirement to ensure stability against overturning failure. Most recent literatures (AASHTO 2012; USACE 2005) require that the design meet only the minimum eccentricity requirements, whereas some recent texts still require meeting both the moment FS and the eccentricity requirements (AASHTO 2002). On the basis of Bowles (1989) designing footing by limiting the eccentricity to the commonly recommended value of $B/6$ is not sufficient to limit base rotation. Bowles (1989) recommends limiting the maximum contact stress such that the ratio of average contact stress for the entire footing to the maximum contact stress for the eccentrically loaded footing will be greater than 0.5. This lateral recommendation has been mostly ignored in practice resulting in unacceptable rotation of many constructed structures.

Before the current LRFD, AASHTO included ASD, which is the same as WSD, and the load factor design (LFD) for bridge foundations. For the LFD, to account for the *Effects of Load Eccentricity*, AASHTO (2002) specifies that footings under eccentric loading shall be designed to ensure that (1) the product of the (ultimate) bearing capacity (of the effective footing) and an appropriate performance factor exceeds the effects of vertical design loads and (2) eccentricity of loading, evaluated based on factored loads, is less than ¼ and ⅜ of the footing dimension in any direction for footings on soil and rock, respectively. *AASHTO LRFD* (AASHTO 2012) specifies the same eccentricity requirements as in the LFD but does not include the other traditionally used requirement, that is, the moment-based FS for overturning design. The first requirement above that states the product of the bearing capacity and an appropriate performance factor exceeds the effects of the maximum bearing pressure is a new requirement and of particular interest.

USACE (2005) states that referring to the analysis performed for the determination of the resultant location for concrete hydraulic structures as the *overturning stability analysis* is a misnomer. It states that a foundation bearing, crushing of the structure toe, and/or a sliding failure will occur before the structure overturns. USACE (2005) refers to the pertinent mode of failure as the rotational failures. However, it recommends using the location of the resultant to ensure rotational stability as presented in Table 8.4.

TABLE 8.4 Requirements for Location of the Resultant—All Structures

Load Conditions	Usual Events	Unusual Events	Extreme Events
Limiting eccentricity	100% of the base in compression (i.e., $e \leq B/6$)	75% of the base in compression (i.e., $e \leq B/4$)	Resultant within base (i.e., $e < B/2$)

Source: Modified after USACE, Stability Analyses of Hydraulic Structures, EM 1110-2-2100, U.S. Army Corps of Engineers, 2005.

Based on USACE (2005), full base contact (i.e., $e \leq B/6$ is specified for usual loading; return period, $T \leq 10$ years) so that there is no chance for uplift pressure to develop in the cracks. In addition, it is also stated that this requirement will help ensure linear behavior for common loading conditions. The limiting eccentricity specified for the unusual loading (10 years $< T \leq 300$ years) permits minor nonlinear behavior. For the extreme loading ($T > 300$ years), it is stated that a shear or bearing failure will occur before overturning (i.e., rotational) failure could occur. As a result, the resultant is permitted to be anywhere within the base. The stability of the structures is said to be ensured by the safety factor requirements for other stability modes, including the limits on the allowable bearing stresses. Although USACE (2005) recognized the correct mechanism involved in the so-called overturning failure of shallow foundation–supported structures, it added more confusion by the specified design requirements and the supporting reasoning.

It is interesting to note that USACE (2005) does not mention the need for limiting the allowable bearing stresses for the usual as well as the unusual loading cases. This, however, is clearly necessary because even if the specified limiting eccentricity conditions are met, the foundation may be subjected to excessive bearing stress, especially near the toe if the vertical load is high. This aspect of the design requirements seems to have been recognized in AASHTO (2002) although the specified requirement that the product of the bearing capacity and an appropriate performance factor exceeds the effects of the maximum bearing pressure is problematic as shown later in this section.

A review of the literature to trace back the original basis for the limiting eccentricity requirements for shallow foundations will assist in the understanding of this issue.

The concept of a limiting eccentricity in the design of shallow foundations originated from the recognition, based on numerous field and laboratory observations, that shallow foundations subjected to lateral loads are susceptible to titling or rotation due to differential settlements in the transverse direction of retaining structure. This differential settlement, and hence tilting, occurs due to nonuniform contact or bearing stress distribution (Lambe and Whitman 1979; Bowles 1982) induced by the load eccentricity.

Tomlinson and Boorman (2001) in discussing the need to calculate settlement for checking the serviceability limit state design pointed out that excessive tilting of the foundation will cause an increase in eccentricity and result in even higher edge pressures. This can lead to yielding or bearing failure of the soils underneath the edge causing the shallow foundation and any supported structure to fail what would appear to be a rotational (or overturning)-type failure. It is important to understand that although the footing or the structure fails in the form for excessive rotation, the soils underneath the footing edge fail in shear or bearing capacity due to excessive bearing stress in compression, as shown in Figure 8.7.

A large eccentricity is neither necessary nor a sufficient cause for soil yielding or bearing failure to occur underneath the footing toe. It is the combination of the eccentricity and the vertical load that exerts higher contact or bearing stress and cause the soil to yield underneath the toe and ultimately resulting in a rotational failure of the structure. More generally, excessive contact bearing stress solely due a concentrically load spread footing (i.e., $e = 0$) causes a bearing failure of the footing in the traditional sense. As the eccentricity increases above zero, bearing stresses becomes progressively more nonuniform leading, and finally depending on the magnitude of the vertical load, to yielding or bearing failure of the soil underneath the edge. A low vertical load is not likely to cause rotational failure even if the eccentricity is very high. In this case, it is more likely a sliding failure will occur. The other extreme is the traditional bearing failure when vertical load is very high, but the eccentricity is either zero or very low.

Note that in Figure 8.8, rotational failure of the footing occurs due to bearing capacity failure in local shear of the soils underneath the toe area due to excessive nonuniform contact stress resulting from eccentric loading. As the footing tilts, its base, at least some part of it, maintains in firm contact with the foundation soils at all times and does not "overturn" by rotating about a point at the toe. Thus, it is necessary to consider the so-called overturning failure of spread footing as a *soil bearing capacity failure in local shear* as discussed below.

The type of footing behavior discussed earlier is similar to the familiar and widely used axial load (P) and moment interaction (M) curve used in determining the capacities of structural elements. The main difference is that soils cannot support tension or flexural loading. Spread footing foundations resist any unbalanced moment by developing nonuniform distribution of the compressive contact or bearing stress. Foundation failures always occur due the failure of soils in shear. Thus, the moment equilibrium methodology, which does not consider the shear failure of soils, is not reliable in ensuring the stability of the footing against rotational failure.

Furthermore, as pointed out by Bowles (1995), for a footing designed to support a concentrically applied vertical service load (P_s) corresponding to a uniform permissible bearing stress (q_{perm}), if due to lateral load the eccentricity increases to $e = B/6$, the maximum bearing stress at toe q_{max} will be equal to $2q_{perm}$ and q_{min} at the will be equal to zero.

The recommendation that the eccentricity (e) for shallow foundations in soils be limited to $B/6$ was based on the postulation that the q_{max} will be limited to two times the q_{perm} and that for this nonuniform bearing stress conditions the resulting rotation or tilting of shallow foundations when founded on competent soils will be within tolerable or permissible limit. Furthermore, if the footing design was controlled by the allowable bearing capacity (q_a) with an FS $\geq$ 3.0, for eccentricity, $e = B/6$, q_{max} will be equal to two times q_a. That is, the FS against bearing capacity failure of the toe is $\geq$1.5, which was considered acceptable against overturning failure. The original and main purpose, however, was to simplify the design so that detailed analysis for evaluating tilting or rotation, which often can be complex, will not be necessary when footings are founded in competent soils. However, to evaluate the estimated or

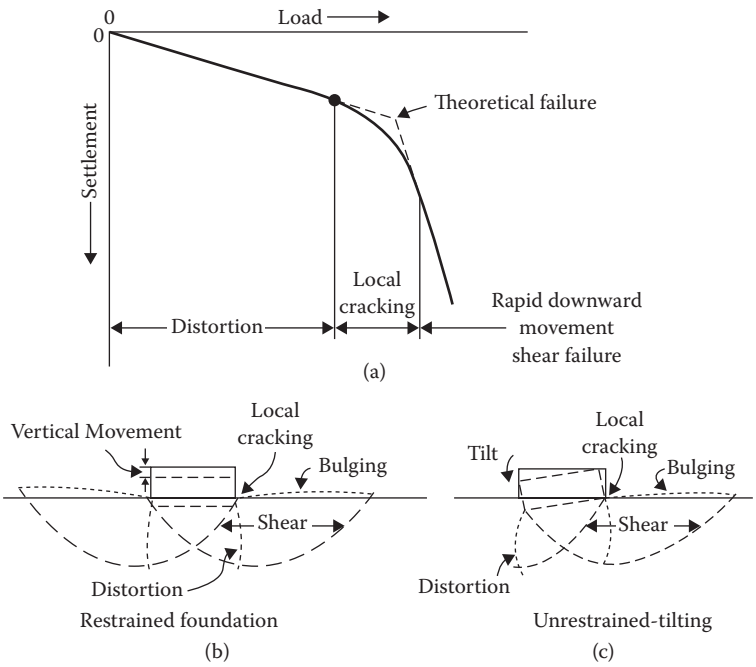


FIGURE 8.8 Bearing capacity failures. (a) Load–settlement curve; (b) shear failures; and (c) rotational ("overturning") failure. (Modified after Sowers, G. F., and Vesic, A. B., *Highway Research Board Bulletin*, No. 342, 1962.)

expected rotation or tilting due to eccentric loading, detailed differential settlement analysis should be performed when foundation soil conditions are highly variable, for critical structure, and footings founded on weak or more compressible soils.

The above limiting eccentricity requirement for footings founded on soils started as a simplified substitute analysis method for evaluating differential settlement or tilting of footing was soon specified as a check against overturning failure. The concept of limiting eccentricity was then extended for footings founded on rock and misleadingly for seismic design (NCHRP 1991; AASHTO 2002) and more recently for each of the three LRFD limit states (AASHTO 2012) for bridge foundation design.

The widely recommended practice of limiting load eccentricity although results in a design with adequate stability against rotation when footings are founded on competent soils, it cannot ensure rotational stability when footings are founded on more compressible soils. Most significantly, the limiting eccentricity requirement is based on limiting differential settlement, which is a service limit state design parameter, Therefore, in LRFD use of the limiting eccentricity requirements must be limited to service limit states design to limit differential settlement of the footing and thus rotation or tilting of the supported structure. Rotational stability under the factored load must be ensured by evaluating the shear failure of the soils due to excessive factored contact or bearing stress for both the strength and the extreme limit states design.

On the basis of the above discussion, stability against both traditional bearing capacity failure and rotational failure will be prevented provided shallow foundations are designed to achieve adequate FS against soil bearing failures evaluated by considering all possible combinations of the total vertical loads and the eccentricities of the resultant loads.

In summary, the requirements that the eccentricity (e) of $B/6$ and $B/4$ are appropriate for use in the WSD and LRFD service limit state design of shallow foundations in competent soils with low compressibility and rock, respectively. The rotational stability evaluation of shallow foundations for seismic and LRFD strength and extreme limit state design should be performed based on the evaluation of the soil bearing stability under the extreme eccentricity and total vertical load combinations. The design must ensure that the maximum factored uniform bearing stress $(q_o)_{max}$ is less than or equal to the factored uniform nominal bearing resistance for all possible combinations of the factored vertical loads and the eccentricities of the factored resultant load.

Finally, it should be noted that tilting failure of spread footing supported structures can also occur due to the global slope stability–type failures. However, the failure mechanism in this case is different than that involved in tilting failure discussed earlier, which occurs due to excessive rotation of the footing due to load eccentric loading. Eccentric loading is not necessary to cause tilting failure associated with the global slope failure and the titling in the case is generally inwards, that is toward the slope or the retained ground. Furthermore, spread footing rotation resulting in the inward tilting of earth retaining structures such as bridge abutments can also occur due to the settlement of the heel when compressible soil are present below the foundation (Bowles 1982).

8.5 Bearing Capacity for Shallow Foundations

8.5.1 Static Bearing Capacity—Theoretical Methods

This section deals with the ultimate bearing (q_{ult}) of shallow foundations under compression loading. The nominal bearing resistance in compression (q_n) in the LRFD is synonymous with q_{ult} in the WSD.

The foundation soils underneath a spread footing subjected to compression loading can be failure in three different shear mechanisms. The corresponding bearing capacity failures are referred to as (1) general shear failure, (2) local shear failure, and (3) punching shear failure as shown in Figure 8.4. The shear mechanism by which a given footing is likely to fail depends mainly on the density of the foundation soils and the footing depth to width ratio (D/B') as shown in Figure 8.9.

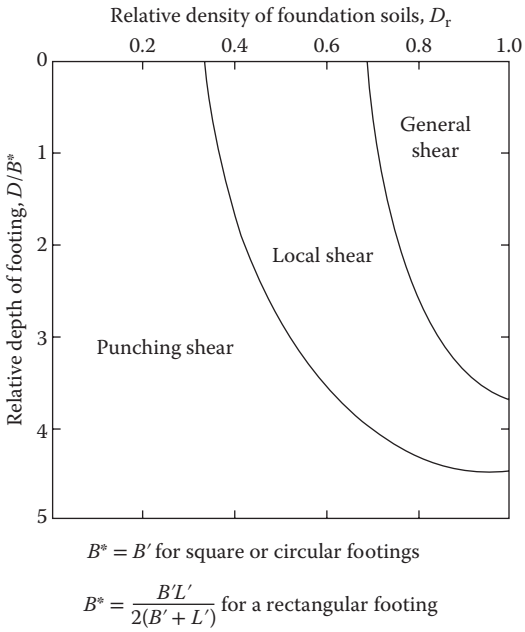


FIGURE 8.9 Modes of bearing capacity failure of spread footing in sand. (After Vesic, A. S., *Foundation Engineering Handbook*, Edited by Winterkorn, H. F. and Fang, H. Y., Van Nostrand Reinhold Company, 1975.)

8.5.1.1 Bearing Capacity Failure in General Shear

The computation of q_{ult} for shallow foundations on soil can be considered as an elastic–plastic kinematic equilibrium problem. However, what hinders us from finding closed form analytical solutions is the difficulty in the selection of a mathematical model of soil constitutive relationships. Bearing capacity theory is still limited to solutions established for the rigid plastic solid of the classic theory of plasticity. Consequently, only approximate methods are currently available for solving the problem. Prandtl (1924) and Reissner (1924) developed kinematic conditions and solutions to the bearing capacity problem by considering a uniform, rigid plastic, weightless soil. Terzaghi and Peck (1948) defined three different zones of plastic failures under a footing experiencing bearing capacity failure in general shear as shown in Figure 8.10.

A strip or continuous footings is a shallow footing with $L'/B' \geq 5$. Based on the work by Prandtl (1920), Reissner (1924), and others, the gross ultimate bearing capacity in compression (q_{ult}) of a centrically loaded strip footing is given by

$$q_{ult} = q_N = cN_c + qN_q + 0.5\gamma B'N_\gamma \tag{8.11}$$

where c is the soil cohesion, q the overburden stress at the level of the footing bottom ($\gamma_e D$), γ_e the unit weight of the soil above the footing bottom, and γ_g the unit weight of the soil below the footing, and B' is the effective width of the strip footing. The parameters N_c, N_q, and N_γ are bearing capacity factors defined as functions of the friction angle of the foundation soils (φ) as presented below.

$$N_q = \text{bearing capacity factor related to surcharge } (\overline{q})$$

$$= \left(e^{\pi \tan\varphi}\right) \tan^2\left(45^o + \frac{\varphi}{2}\right) \tag{8.12}$$

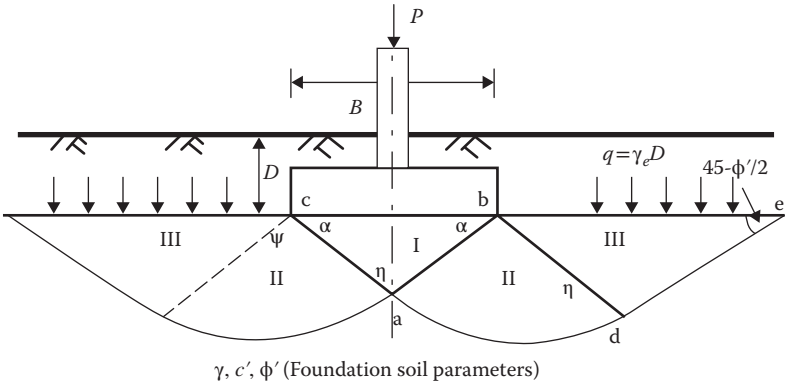


FIGURE 8.10 General shear failures.

$$N_c = \text{bearing capacity factor related to cohesion } (c)$$

$$= (N_q - 1)\cot\varphi, \quad \text{for } \varphi > 0$$

$$= (2 + \pi) = 5.14, \quad \text{for } \phi = 0.0, \text{ and}$$

$$N_\gamma = \text{bearing capacity factor related to soil unit weight } (\gamma)$$

$$= 2\left(N_q + 1\right)\tan\varphi$$

The three N factors are used to represent the influence of the cohesion (N_c), unit weight (N_γ), and overburden pressure (N_q) of the soil on bearing capacity. The above bearing capacity factor N_γ was developed by Caquot and Kerisel (1948). Values of the above bearing capacity factors are presented in Table 8.5 and Figure 8.11.

For long-term analyses, the friction of the above bearing capacity equations is the effective or drained friction angle (φ' and $c = c'$, where c' is the effective cohesion that in general should be taken as equal to 0.0 for soils. In this case, the term q in the second term of the Equation 8.11 is the effective overburden stress (q') at the bottom of the footing. Thus for long-term bearing capacity, the ultimate bearing capacity equation for strip footing founded on soil is given by

$$q_{ult} = q_N = q'N_q + 0.5\gamma B'N_\gamma \tag{8.13}$$

If the strip footing is founded on fine-grained or clay soils, the short-term ultimate bearing capacity may be obtained from Equation 8.11 by replacing cohesion (c) with the soil undrained shear strength (s_u), and $N_c = 5.14$, $N_\gamma = 0.0$ $N_q = 1.0$ from Figure 8.11 because in this case the friction angle, $\varphi = 0.0$. That is,

$$q_{ult} = q_N = s_u N_c + qN_q \tag{8.14a}$$

$$q_{ult} = q_N = 5.14\ S_u + q \tag{8.14b}$$

In the Equation 8.14, q is the total overburden stress at the bottom of the footing.

Once the ultimate gross bearing capacity or the nominal bearing resistance in compression is known, the unit gross allowable bearing capacity (q_a) as well as the unit factored nominal resistance (q_R) can be determined as follows:

$$q_a = \frac{q_{ult}}{FS} \tag{8.15}$$

and

$$q_R = \phi_b q_N \tag{8.16}$$

TABLE 8.5 Values of the Bearing Capacity Factors in Equation 8.13

φ	N_c	N_q	N_γ	φ	N_c	N_q	N_γ
0	5.14	1.00	0.00	26	22.25	11.85	12.54
1	5.38	1.09	0.07	27	23.94	13.20	14.47
2	5.63	1.20	0.15	28	25.80	14.72	16.72
3	5.90	1.31	0.24	29	27.86	16.44	19.34
4	6.19	1.43	0.34	30	30.14	18.40	22.40
5	6.49	1.57	0.45	31	32.67		25.99
6	6.81	1.72	0.57	32	35.49	23.18	30.22
7	7.16	1.88	0.71	33	38.64	26.09	35.19
8	7.53	2.06	0.86	34	42.16	29.44	41.06
9	7.92	2.25	1.03	35	46.12	33.30	48.03
10	8.35	2.47	1.22	36	50.59	37.75	56.31
11	8.80	2.71	1.44	37	55.63	42.92	66.19
12	9.28	2.97	1.69	38	61.35	48.93	78.03
13	9.81	3.26	1.97	39	67.87	55.96	92.25
14	10.37	3.59	2.29	40	75.31	64.20	109.41
15	10.98	3.94	2.65	41	83.86	73.90	130.22
16	11.63	4.34	3.06	42	93.71	85.38	155.55
17	12.34	4.77	3.53	43	105.11	99.02	186.54
18	13.10	5.26	4.07	44	118.37	115.31	224.64
19	13.93	5.80	4.68	45	133.88	134.88	271.76
20	14.83	6.40	5.39	46	152.10	158.51	330.35
21	15.82	7.07	6.20	47	173.64	187.21	403.67
22	16.88	7.82	7.13	48	199.26	222.31	496.01
23	18.05	8.66	8.20	49	229.93	265.51	613.16
24	19.32	9.60	9.44	50	266.89	319.07	762.89
25	20.72	10.66	10.88	—	—	—	—

Source: Data from AASHTO, *AASHTO LRFD Bridge Design Specifications*, Customary U.S. Units, 2012, American Association of State Highway and Transportation Officials, Washington D.C., 2012.

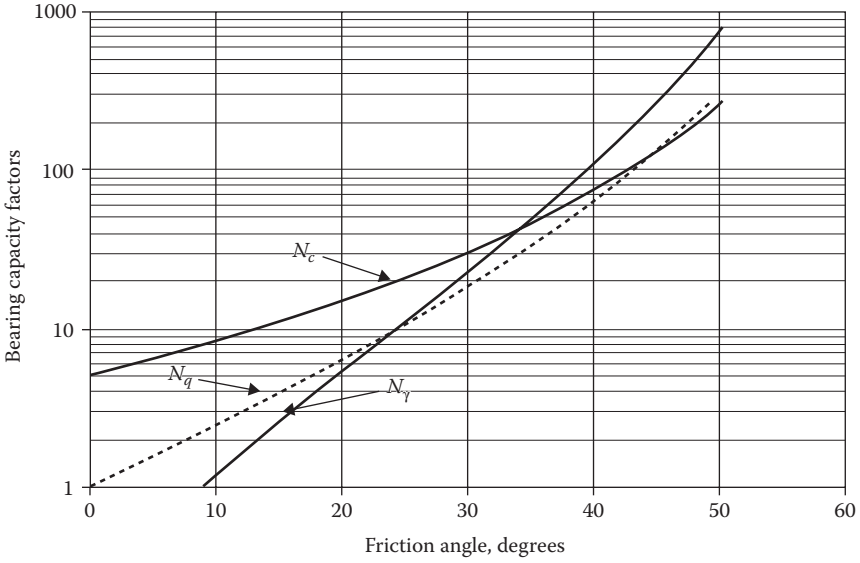


FIGURE 8.11 Bearing capacity factors N_c, N_q, and N_γ as functions of the friction angle.

For simplicity, the q_{ult} will be used in the rest of this chapter to represent both the ultimate bearing capacity for use in WSD and the nominal bearing resistance in compression (q_N) for use in LRFD.

One of the well-known more generalized bearing capacity equations applicable to strip, round as well as rectangular footings, is the well-known Terzaghi's equation (Terzaghi 1943), which can be written as

$$q_{ult} = cN_c s_c + \overline{q}N_q s_q + 0.5\gamma B' N_\gamma s_\gamma \tag{8.17}$$

The values of the N parameters for use in Equation 8.17 are provided in Table 8.6.

The factors, s_c, s_q, and s_γ are shape correction factors as defined below.

- For free draining coarse-grained soils and long-term or drained bearing capacity in saturated fine-grained soils (i.e., when $c' = 0.0$),

$$s_c = 1 + \left(\frac{B'}{L'}\right)\left(\frac{N_q}{N_c}\right), \quad \frac{s_q = 1 + (B'/L')\tan\phi}{s_\gamma = 1 - 0.4(B'/L')} \tag{8.18}$$

- For short-term or undrained bearing capacity in saturated fine-grained soils (i.e., $\phi = 0$),

$$s_c = 1 + \left(\frac{B'}{5L'}\right), \quad s_q = 1, \quad \text{and } s_\gamma = 1 \tag{8.19}$$

The values of the shape correction parameters s_c and s_r for strip, round, and square footings are presented in Table 8.7.

The bearing capacity equations presented earlier are valid only for the general shear failure case shown in Figures 8.3 and 8.10. The assumptions used in the development of these equations include the following:

- The footing base is rough and the soil beneath the base is incompressible, which implies that the wedge abc (zone I) in Figure 8.10 is no longer an active Rankine zone but is in an elastic state. Consequently, zone I must move together with the footing base.

TABLE 8.6 Bearing Capacity Factors for the Terzaghi Equation 8.17

φ (degree)	N_c	N_q	N_γ	$K_{p\gamma}$
0	5.7[a]	1.0	0	10.8
5	7.3	1.6	0.5	12.2
10	9.6	2.7	1.2	14.7
15	12.9	4.4	2.5	18.6
20	17.7	7.4	5.0	25.0
25	25.1	12.7	9.7	35.0
30	37.2	22.5	19.7	52.0
34	52.6	36.5	36.0	—
35	57.8	41.4	42.4	82.0
40	95.7	81.3	100.4	141.0
45	172.3	173.3	297.5	298.0
48	258.3	287.9	780.1	—
50	347.5	415.1	1153.2	800.0

Source: After Bowles, J. E., *Foundation Analysis and Design*, Fifth Edition, McGraw-Hill Companies, Inc., 1996.

Note: Values of N_γ for φ of 0°, 34°, and 48° are original Terzaghi values and used to back-compute $K_{p\gamma}$.

[a] $N_c = 1.5\pi + 1$ (Terzaghi 1943, p. 127).

TABLE 8.7 Shape Factors for the Terzaghi Equation 8.17

	Footing Shape		
Shape Factor	Strip	Round	Square
s_c	1.0	1.3	1.3
s_γ	1.0	0.6	0.8

Source: After Terzaghi, K., *Theoretical Soil Mechanics*, John Wiley & Sons, New York, 1943.

- Zone II in Figure 8.10 is an immediate state lying on a log spiral arc ad.
- Zone III is a passive Rankine zone and is in a plastic state bounded by a straight line ed.
- The shear resistance along bd is neglected because the equation was intended for footings where $D < B'$.

Meyerhof (1951, 1963), Hansen (1970), and Vesic (1973, 1975) further extended Terzaghi's bearing capacity equation by including footing shape factor (s_i), footing embedment depth factor (d_i), load inclination factor (i_i), sloping ground factor (g_i), and tilted base factor (b_i). Chen (1975) reevaluated N factors in Terzaghi's equation using limit analysis method. These efforts resulted in significant extensions of Terzaghi's bearing capacity equation. The general form of the bearing capacity equation (Hansen 1970; Vesic 1973, 1975) can be expressed as

$$q_{\text{ult}} = cN_c s_c d_c i_c g_c b_c + \bar{q} N_q s_q d_q b_q + 0.5\gamma B' N_\gamma s_\gamma d_\gamma i_\gamma g_\gamma b_\gamma \tag{8.20}$$

Values of bearing capacity factors N_c, N_q, and N_γ can be found in Table 8.8. Values of the other factors are summarized in Table 8.9.

For saturated cohesive soils under short-term conditions, $c = s_u$ and $\varphi = 0$. For the ultimate short-term bearing capacity of spread footing founded in this type of soil, Equation 8.20 reduces to

$$q_{\text{ult}} = 5.14 s_{\text{u}} (1 + s'_c + d'_c - i'_c - b'_c - g'_c) + \bar{q} \tag{8.21}$$

where s_{u} is the undrained shear strength of cohesive soils.

As shown in Table 8.8, N_c and N_q are the same as proposed by Meyerhof (1963), Hansen (1970), Vesic (1973), or Chen (1975). Nevertheless, there is a wide range of values for N_γ as suggested by different authors. Meyerhof (1963) and Hansen (1970) use the plain-strain value of φ, which may be up to 10% higher than those obtained from the conventional triaxial tests. Vesic (1975) argued that a shear failure in soil under the footing is a process of progressive rupture at variable stress levels, and an average mean normal stress should be used for bearing capacity computations. Another reason causing the differences in the N_γ value is how to evaluate the impact of the soil compressibility on bearing capacity computations. The value of N_γ still remains controversial because rigorous theoretical solutions are not available. In addition, comparisons of predicted solutions against model footing test results are inconclusive.

8.5.1.2 Local and Punching Shear Failures

The bearing capacity equations presented above are applicable to the footings when the failure mode is one of the general shear failure types. On the basis of Terzaghi (1943), the above bearing capacity equation may be used to evaluate the bearing capacity footings that is predicted to fail in one of the other two modes of shear failures shown in Figure 8.3 provided the shear strength parameters c and φ of the foundation soil are reduced as follows:

$$c^* = 0.67c \tag{8.22}$$

$$\varphi^* = \tan^{-1}(0.67 \tan \varphi) \tag{8.23}$$

TABLE 8.8 Bearing Capacity Factors for Equation 8.20

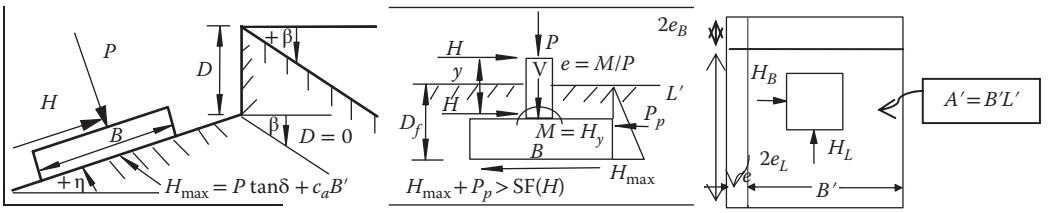
φ	N_c	N_q	$N_{\gamma(M)}$	$N_{\gamma(H)}$	$N_{\gamma(V)}$	$N_{\gamma(C)}$	N_q/N_c	tan φ
0	5.14	1.00	0.00	0.00	0.00	0.00	0.19	0.00
1	5.38	1.09	0.00	0.00	0.07	0.07	0.20	0.02
2	5.63	1.20	0.01	0.01	0.15	0.16	0.21	0.03
3	5.90	1.31	0.02	0.02	0.24	0.25	0.22	0.05
4	6.18	1.43	0.04	0.05	0.34	0.35	0.23	0.07
5	6.49	1.57	0.07	0.07	0.45	0.47	0.24	0.09
6	6.81	1.72	0.11	0.11	0.57	0.60	0.25	0.11
7	7.16	1.88	0.15	0.16	0.71	0.74	0.26	0.12
8	7.53	2.06	0.21	0.22	0.86	0.91	0.27	0.14
9	7.92	2.25	0.28	0.30	1.03	1.10	0.28	0.16
10	8.34	2.47	0.37	0.39	1.22	1.31	0.30	0.18
11	8.80	2.71	0.47	0.50	1.44	1.56	0.31	0.19
12	9.28	2.97	0.60	0.63	1.69	1.84	0.32	0.21
13	9.81	3.26	0.74	0.78	1.97	2.16	0.33	0.23
14	10.37	3.59	0.92	0.97	2.29	2.52	0.35	0.25
15	10.98	3.94	1.13	1.18	2.65	2.94	0.36	0.27
16	11.63	4.34	1.37	1.43	3.06	3.42	0.37	0.29
17	12.34	4.77	1.66	1.73	3.53	3.98	0.39	0.31
18	13.10	5.26	2.00	2.08	4.07	4.61	0.40	0.32
19	13.93	5.80	2.40	2.48	4.68	5.35	0.42	0.34
20	14.83	6.40	2.87	2.95	5.39	6.20	0.43	0.36
21	15.81	7.07	3.42	3.50	6.20	7.18	0.45	0.38
22	16.88	7.82	4.07	4.13	7.13	8.32	0.46	0.40
23	18.05	8.66	4.82	4.88	8.20	9.64	0.48	0.42
24	19.32	9.60	5.72	5.75	9.44	11.17	0.50	0.45
25	20.72	10.66	6.77	6.76	10.88	12.96	0.51	0.47
26	22.25	11.85	8.00	7.94	12.54	15.05	0.53	0.49
27	23.94	13.20	9.46	9.32	14.47	17.49	0.55	0.51
28	25.80	14.72	11.19	10.94	16.72	20.35	0.57	0.53
29	27.86	16.44	13.24	12.84	19.34	23.71	0.59	0.55
30	30.14	18.40	15.67	15.07	22.40	27.66	0.61	0.58
31	32.67	20.63	18.56	17.69	25.99	32.33	0.63	0.60
32	35.49	23.18	22.02	20.79	30.21	37.85	0.65	0.62
33	38.64	26.09	26.17	24.44	35.19	44.40	0.68	0.65
34	42.16	29.44	31.15	28.77	41.06	52.18	0.70	0.67
35	46.12	33.30	37.15	33.92	48.03	61.47	0.72	0.70
36	50.59	37.75	44.43	40.05	56.31	72.59	0.75	0.73
37	55.63	42.92	53.27	47.38	66.19	85.95	0.77	0.75
38	61.35	48.93	64.07	56.17	78.02	102.05	0.80	0.78
39	67.87	55.96	77.33	66.75	92.25	121.53	0.82	0.81
40	75.31	64.19	93.69	79.54	109.41	145.19	0.85	0.84
41	83.86	73.90	113.98	95.05	130.21	174.06	0.88	0.87
42	93.71	85.37	139.32	113.95	155.54	209.43	0.91	0.90
43	105.11	99.01	171.14	137.10	186.53	253.00	0.94	0.93
44	118.37	115.31	211.41	165.58	224.63	306.92	0.97	0.97
45	133.87	134.87	262.74	200.81	271.74	374.02	1.01	1.00
46	152.10	158.50	328.73	244.64	330.33	458.02	1.04	1.04
47	173.64	187.20	414.32	299.52	403.65	563.81	1.08	1.07
48	199.26	222.30	526.44	368.66	495.99	697.93	1.12	1.11
49	229.92	265.49	674.91	456.40	613.13	869.17	1.15	1.15
50	266.88	319.05	873.84	568.56	762.85	1089.46	1.20	1.19

Source: Data from Meyerhof, G. G., *Canadian Geotechnical Journal*, **I**, No. 1, pp.16–26, 1963; Hansen, B. J., *A Revised and Extended Formula for Bearing Capacity*, Bulletin No. 28, Danish Geotechnical Institute, Copenhagen, pp. 5–11, 1970; Vesic, A. S., *Foundation Engineering Handbook*, Edited by Winterkorn, H. F. and Fang, H. Y., Van Nostrand Reinhold Company, 1975; and Chen, W. F., *Limit Analysis and Soil Plasticity*, Elsevier, Amsterdam, the Netherlands, 1975.

Note: N_c and N_q are the same for all four methods; subscripts identify author for N_γ.

TABLE 8.9 Footing Shape, Footing Depth, Load Inclination, Foundation Ground (Slope) Inclination (Slope), and Footing Base Inclination Factors for Equations 8.20 and 8.21

Shape Factors	Depth Factors
$s_c = 1.0 + \frac{N_q}{N_c}\frac{B'}{L'}$ $s_c = 1.0$ (for strip footing)	$d_c = 1.0 + 0.4k \begin{cases} k = \frac{D}{B'} & \text{for } \frac{D}{B'} \le 1 \\ k = \tan^{-1}\left(\frac{D}{B'}\right) & \text{for } \frac{D}{B'} > 1 \end{cases}$
$s_q = 1.0 + \frac{B'}{L'}\tan\phi$ (for all ϕ)	$d_q = 1 + 2\tan\phi(1-\sin\phi)^2 k$ (k defined above)
$s_\gamma = 1.0 - 0.4\frac{B'}{L'} \ge 0.6$	$d_\gamma = 1.00$ (for all ϕ)
Inclination Factors	**Ground Factors (Base on Slope)**
$i'_c = 1 - \frac{mHi}{A'c_a N_c}(\phi = 0)$	$g'_c = \frac{\beta}{5.14}\beta$ in radians ($\phi = 0$)
$i_c = i_q - \frac{1-i_q}{N_q - 1}(\phi > 0)$	$g_c = i_q - \frac{1-i_q}{5.14\tan\phi}$ ($\phi > 0$)
$i_q = \left[1.0 - \frac{H}{P + A'c_a\cot\phi}\right]^m$	$g_q = g_\gamma = (1.0 - \tan\beta)^2$ (for all ϕ)
$i_\gamma = \left[1.0 - \frac{H}{P + A'c_a\cot\phi}\right]^{m+1}$	**Base Factors (Tilted Base)**
$m = m_B = \frac{2 + B'/L'}{1 + B'/L'}$ or	$b'_c = g'_c$ ($\phi = 0$)
$m = m_L = \frac{2 + L'/B'}{1 + L'/B'}$	$b_c = 1 - \frac{2\beta}{5.14\tan\phi}$ ($\phi > 0$)
	$b_q = b_\gamma = (1.0 - \eta\tan\phi)^2$ (for all ϕ)



Source: After Vesic, A. S., *ASCE Journal of the Soil Mechanics and Foundation Engineering Division*, **99**, No. SM1, pp. 45–73, 1973; and Vesic, A. S., *Foundation Engineering Handbook*, Edited by Winterkorn, H. F. and Fang, H. Y., Van Nostrand Reinhold Company, 1975.

Notes:

1. When $\varphi = 0$ (and $\beta \neq 0$) use $N_\gamma = -2\sin(\pm\beta)$ in N_γ term.
2. Compute $m = m_B$ when $H_i = H_B$ (H parallel to B) and $m = m_L$ when $H_i = H_L$ (H parallel to L). For both H_B and H_L use $m = \sqrt{m_B^2 + m_L^2}$.
3. $0 \le i_q,\ i_\gamma \le 1$
4. $\beta + \eta \le 90°;\ \beta \le \phi$

where

A' = effective footing dimension as shown in Figure 8.12
D = depth from ground surface to base of footing
P = vertical load on footing
H = horizontal component of load on footing with $H_{max} \le P\tan\delta + c_a A'$
c_a = adhesion to base ($0.6c \le c_a \le 1.0c$)
δ = friction angle between base and soil ($0.5\varphi \le \delta \le \varphi$)
β = slope of ground away from base with (+) downward
η = tilt angle of base from horizontal with (+) upward $2e_B$

Vesic (1975) suggested that the above reduction of φ is too conservative. He proposed the following equation for a reduction factor for cohesionless soils that varies with the relative density D_r:

$$\phi^* = \tan^{-1}[(0.67 + D_r - 0.75D_r^2)\tan\phi] \quad (\text{for } 0 < D_r < 0.67) \tag{8.24}$$

8.5.1.3 Effects of Ground Water Table

Ultimate bearing capacity should be estimated considering the highest anticipated ground water table. When groundwater is present with the equivalent effective unit weight γ_e, as defined below, it should be used in evaluating the overburden stress $\overline{q}$ used with N_q and also to replace γ used with N_γ. As illustrated in Figure 8.12, the weighted average unit weight for the *0.5γB* term can be determined as follows:

$$\gamma = \begin{cases} \gamma_{avg} & \text{for } d_w \geq B' \\ \gamma' + (d_w/B')(\gamma_{avg} - \gamma') & \text{for } 0 < d_w < B' \\ \gamma' & \text{for } d \leq 0 \end{cases} \tag{8.25}$$

8.5.1.4 Effects of Eccentric Load

As stated earlier, for footings with eccentricity, effective footing size is determined as follows:

$$A' = B' \times L' \tag{8.26}$$

where, $B' = B - 2e_L$, and $L' = L - 2e_B$. Refer to Figures 8.1, 8.13, and 8.14 for loading definitions and footing dimensions. For example, the actual distribution of contact pressure along the *L*-direction for a rigid footing with eccentricity e_B about the *B*-axis may be obtained as follows:

$$q_{\substack{\max \\ \min}} = P(1 \pm 6e_B/L)/BL \quad (\text{for } e_B < L/6) \tag{8.27}$$

$$q_{\substack{\max \\ \min}} = \begin{cases} \dfrac{2P}{3B(L/2 - e_B)} & (\text{for } L/6 < e_B < L/2) \\ 0 \end{cases} \tag{8.28}$$

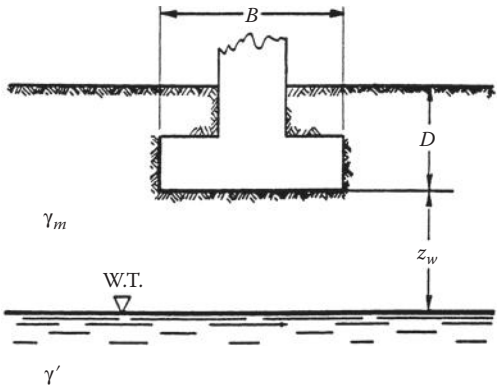


FIGURE 8.12 Influence of ground water table on bearing capacity. (After AASHTO, *Standard Specifications for Highway Bridges,* Seventeenth Edition, American Association of State Highway and Transportation Officials, Washington, DC, 2002.)

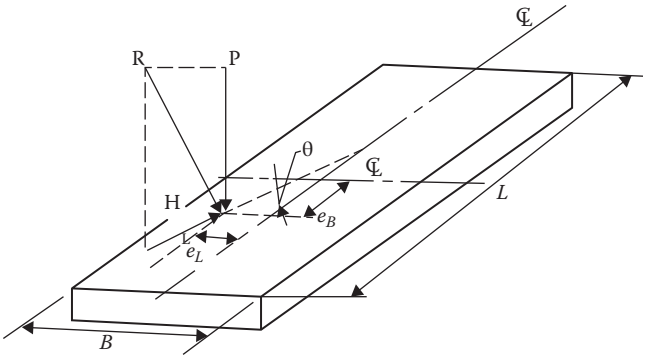


FIGURE 8.13 Method of computing effective footing dimensions for eccentrically loaded footing. (After AASHTO, *Standard Specifications for Highway Bridges,* Seventeenth Edition, American Association of State Highway and Transportation Officials, Washington, DC, 2002.)

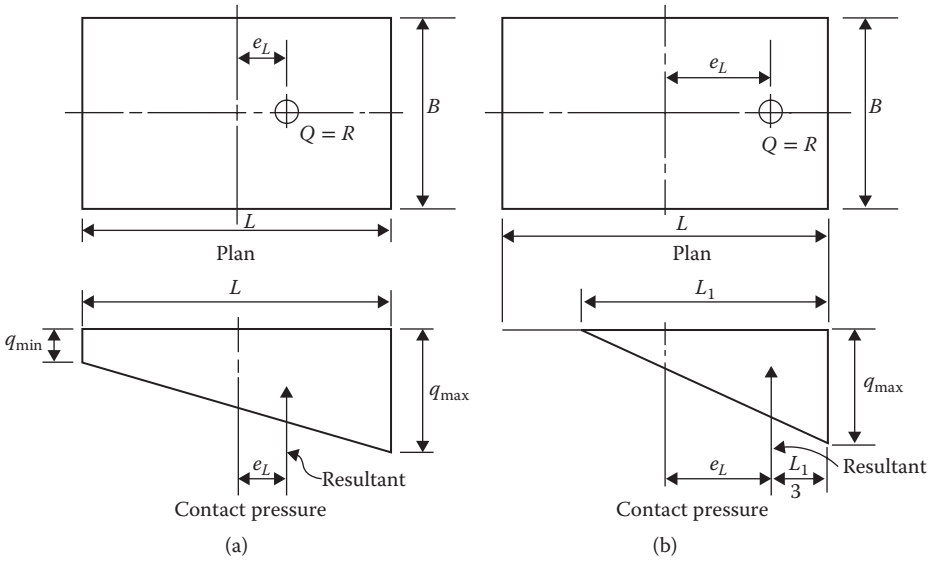


FIGURE 8.14 Contact pressure for footing loaded eccentrically about one axis. (a) For $e_L \le \frac{L}{6}$; and (b) For $\frac{L}{6} < e_L < \frac{L}{2}$. (After AASHTO, *Standard Specifications for Highway Bridges*, Seventeenth Edition, American Association of State Highway and Transportation Officials, Washington, DC, 2002.)

Contact pressure for footings with eccentric loading in the *B* direction may be determined using the above equations by replacing terms *L* with *B* and terms *B* with *L*, respectively. For an eccentricity in both the directions, reference is available in AASHTO (2002 and 2012).

8.6 Static Bearing Capacity—Empirical Methods

8.6.1 Based on Standard Penetration Tests (SPT)

Terzaghi and Peck (1948, 1967) proposed a method using SPT blow counts to estimate ultimate bearing capacity for footings on sand. Modified by Peck et al. (1974), this method is presented in the form of the chart shown in Figure 8.15. For a given combination of footing width and SPT blow counts, the chart can be used to determine the ultimate bearing pressure associated with 25.4 mm (1.0 in.) settlement.

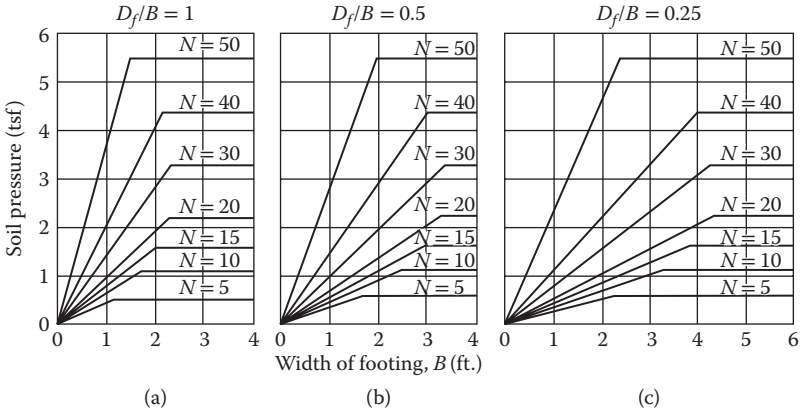


FIGURE 8.15 Design chart for proportioning shallow footings on sand. (After Peck, R. B. et al., *Foundation Engineering*, Second Edition, John Wiley & Sons, Inc., 1974.)

The design chart applies to shallow footings ($D_f \leq B'$ sitting on sand with water table at great depth. The N values used in this figure should be $(N_1)_{60}$, that is, the energy corrected to correspond to 60% of the theoretical hammer energy and normalized to an effective overburden stress of 1.0 tsf (see Skempton 1986).

Similarly, Meyerhof (1956) published the following formula for estimating ultimate bearing capacity using SPT blow counts:

$$q_{\text{ult}} = \left(N'_{\text{avg}}\right)_{60} \frac{B'}{10}\left(C_{w1} + C_{w2}\frac{D_f}{B'}\right)R_I \tag{8.29}$$

where, R_I is load inclination factor shown in Table 8.10 ($R_I = 1.0$ for vertical loads). C_{w1} and C_{w2} are correction factors whose values depend on the position of the water table:

$$\begin{cases} C_{w1} = C_{w2} = 0.5 & \text{for } D_w = 0 \\ C_{w1} = C_{w2} = 1.0 & \text{for } D_w \geq D_f + 1.5B' \\ C_{w1} = 0.5 \text{ and } C_{w2} = 1.0 & \text{for } D_w = D_f \end{cases} \tag{8.30}$$

where $\left(N'_{\text{avg}}\right)_{60}$ is the energy corrected average value of the measured corrected SPT blow counts, which is determined within the range of depths from footing base to $1.5B'$ below the footing. In very fine or silty saturated sand, the measured SPT blow count (N) is corrected for submergence effect as follows:

$$N' = 15 + 0.5(N - 15) \quad \text{for } N > 15 \tag{8.31}$$

8.6.2 Based on Cone Penetration Tests (CPT)

Meyerhof (1956) proposed a relationship between ultimate bearing capacity and cone penetration resistance in sands.

$$q_{\text{ult}} = q_c \frac{B'}{40}\left(C_{w1} + C_{w2}\frac{D_f}{B'}\right)R_I \tag{8.32}$$

where q_c is average value of cone penetration resistance measured at depths from footing base to $1.5B'$ below the footing base. C_{w1}, C_{w2}, and R_I are the same as those defined in Equation 8.29.

Schertmann (1978) recommended correlated values of ultimate bearing capacity to cone penetration resistance in clays as shown in Table 8.11.

TABLE 8.10 Load Inclination Factor (R_I)

For Square Footings

$\frac{H}{P}$	Load Inclination Factor (R_I)		
	$D/B' = 0$	$D/B' = 1$	$D/B' = 3$
0.10	0.75	0.80	0.85
0.15	0.65	0.75	0.80
0.20	0.55	0.65	0.70
0.25	0.50	0.55	0.65
0.30	0.40	0.50	0.55
0.35	0.35	0.45	0.50
0.40	0.30	0.35	0.45
0.45	0.25	0.30	0.40
0.50	0.20	0.25	0.30
0.55	0.15	0.20	0.25
0.60	0.10	0.15	0.20

For Rectangular Footings

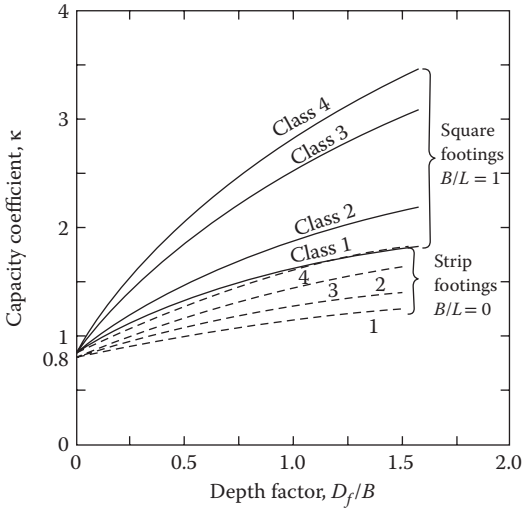
$\frac{H}{P}$	Load Inclination Factor (R_I)					
	$D/B' = 0$	$D/B' = 1$	$D/B' = 5$	$D/B' = 0$	$D/B' = 1$	$D/B' = 5$
0.10	0.70	0.75	0.80	0.80	0.85	0.90
0.15	0.60	0.65	0.70	0.70	0.80	0.85
0.20	0.50	0.60	0.65	0.65	0.70	0.75
0.25	0.40	0.50	0.55	0.55	0.65	0.70
0.30	0.35	0.40	0.50	0.50	0.60	0.65
0.35	0.30	0.35	0.40	0.40	0.55	0.60
0.40	0.25	0.30	0.35	0.35	0.50	0.55
0.45	0.20	0.25	0.30	0.30	0.45	0.50
0.50	0.15	0.20	0.25	0.25	0.35	0.45
0.55	0.10	0.15	0.20	0.20	0.30	0.40
0.60	0.05	0.10	0.15	0.15	0.25	0.35

Source: After Barker, R. M. et al., *Manuals for the Design of Bridge Foundations*, National Cooperative Highway Research Program Report 343, Transportation Research Board, National Research Council, Washington, D.C., 1991.

TABLE 8.11 Correlation between Uniform Ultimate Bearing Capacity (q_{ult}) or Nominal Bearing Resistance in Compression (q_N) and Cone Penetration Resistance (q_c)

	q_{ult} or q_N (ton/ft.^{2})	
q_c (kg/cm^2 or ton/ft.^{2})	Strip Footings	Square Footings
10	5	9
20	8	12
30	11	16
40	13	19
50	15	22

Source: After Schertmann, J. H., *Federal Highway Administration*, Report FHWA-TS-78-209, 1978; and Awkati (1970) Unpublished work as cited in Schmertmann (1978).



Soil Type	Consistency or Density	$(P–P_0)$ (tsf)	Class
Clay	Soft to very firm	<12	1
	Stiff	8–40	2
Sand and gravel	Loose	4–8	2
	Medium to dense	10–20	3
	Very dense	30–60	4
Silt	Loose to medium	<7	1
	Dense	12–30	2
Rock	Very low strength	10–30	2
	Low strength	30–60	3
	Medium to high Strength	$60–100^+$	4

FIGURE 8.16 Values of empirical capacity coefficient, κ. (After Canadian Geotechnical Society 1985.)

8.6.3 Based on Pressuremeter Tests (PMT)

Menard (1965), Baguelin et al. (1978), and Briaud (1986, 1992) proposed using the limit pressure measured in PMT to estimate ultimate bearing capacity.

$$q_{\text{ult}} = r_0 + \kappa(p_l - p_0) \tag{8.33}$$

where r_0 is initial total vertical pressure at foundation level, κ dimensionless bearing capacity coefficient from Figure 8.16, p_l limit pressure measured in PMT at depths from $1.5B'$ above to $1.5B'$ below foundation level, and p_0 total horizontal pressure at the depth where the PMT is performed.

8.7 Presumptive Static Allowable Bearing Pressures

Recommendations for allowable bearing stress $(q_a)_s$ of shallow foundations are available in most of building codes, as presented in Table 8.12. Presumptive value of allowable bearing stress for spread footings are intended for preliminary design when site-specific investigation is not justified. Presumptive allowable bearing stresses usually do not reflect the size, shape, and depth of footing and local water table. Therefore, footing design using such a procedure could be either overly conservative in some cases or unsafe in other cases (Barker et al. 1991). Recommended practice is to use presumptive allowable bearing stresses for preliminary footing sizing and finalize the design using one of the more reliable methods discussed in the preceding sections.

8.8 Seismic Bearing Capacity

Theoretical works by many researchers including Richards et al. (1993), Budhu and Al-Karni (1993), Dormieux and Pecker (1995), Paolucci and Pecker (1997), Kumar and Rao (2003), and Choudhury and Rao (2006) show significant reduction in bearing capacity of spread footings when subjected to seismic loading. This reduction was due to the inertial forces in the foundation soil due to a horizontal seismic acceleration of $k_h g$, where k_h is the coefficient of horizontal seismic acceleration and g is the acceleration due to gravity, and a horizontal load (T) on the foundation due to the inertial forces in the supported structure. In general, any reduction in the seismic bearing capacity due to the inertial forces in the foundation soils is relatively minor and may be neglected. The majority of the reduction occurs due to

TABLE 8.12 Presumptive Allowable Bearing Pressures for Spread Foundations

Type of Bearing Earth Material	In-Place Conditions	q_{all} (ton/ft.²) Range	q_{all} (ton/ft.²) Recommended Value for Use
Massive crystalline igneous and metamorphic rock: granite, diorite, basalt, gneiss, thoroughly cemented conglomerate (sound condition allows minor cracks)	Hard, sound rock	60–100	80
Foliated metamorphic rock: slate, schist (sound condition allows minor cracks)	Medium hard sound rock	30–40	35
Sedimentary rock: hard cemented shales, siltstone, sandstone, limestone without cavities	Medium hard sound rock	15–25	20
Weathered or broken bed rock of any kind except highly argillaceous rock (shale); rock quality designation <25	Soft rock	8–12	10
Compaction shale or other highly argillaceous rock in sound condition	Soft rock	8–12	10
Well graded mixture of fine and coarse-grained soil: glacial till, hardpan, boulder clay (GW-GC, GC, SC)	Very compact	8–12	10
Gravel, gravel–sand mixtures, boulder–gravel mixtures (SW, SP)	Very compact	6–10	7
	Medium to compact	4–7	5
	Loose	2–6	3
Coarse to medium sand, sand with little gravel (SW, SP)	Very compact	4–6	4
	Medium to compact	2–4	3
	Loose	1–3	1.5
Fine to medium sand, silty or clayey medium to coarse sand (SW, SM, SC)	Very compact	3–5	3
	Medium to compact	2–4	2.5
	Loose	1–2	1.5

Source: Modified from NAVFAC, Foundations and Earth Structures, DM 7.02, Naval Facilities Engineering Command, Alexandria, VA, 1986.

Notes:

1. Presumptive allowable bearing pressures are based on allowable footing settlement. For working stress design, limiting net uniform contact stress (q'_0) to these presumptive allowable pressures may provide a reasonable, but known, factor of safety against bearing capacity failure.
2. If fine-grained soils, organic soils, collapsible or swelling soils, very loose cohesionless soils, or uncompacted fill soils are present within the depth of influence of the footing, which can vary from $3B'$ to $5B'$ below the bottom of the footing, site- and project-specific investigation is required to determine q_{all}.
3. If tabulated recommended values for q_{all} for rock exceed measured unconfined compressive strength of intact rock specimen, limit q_{all} to the measured unconfined compressive strength.
4. The tabulated values were developed and recommended for building structures and should only be used for preliminary design of spread footings for bridge structures.
5. Variations of q_{all} for size, depth, and arrangement of footings are given in Table 2 of NAVFAC (1986).

the inclination of the resultant applied load induced by the horizontal load H. In this case, the seismic bearing capacity $(q_{ult})_{seismic}$ or the nominal resistance in compression $(q_N)_{seismic}$ may be taken as that for the static loading, that is, q_{ult} or q_N of the same footing effective width (B′) provided the theoretical equation are used and potential effects of the load inclination is considered in the evaluation of q_{ult} or q_N.

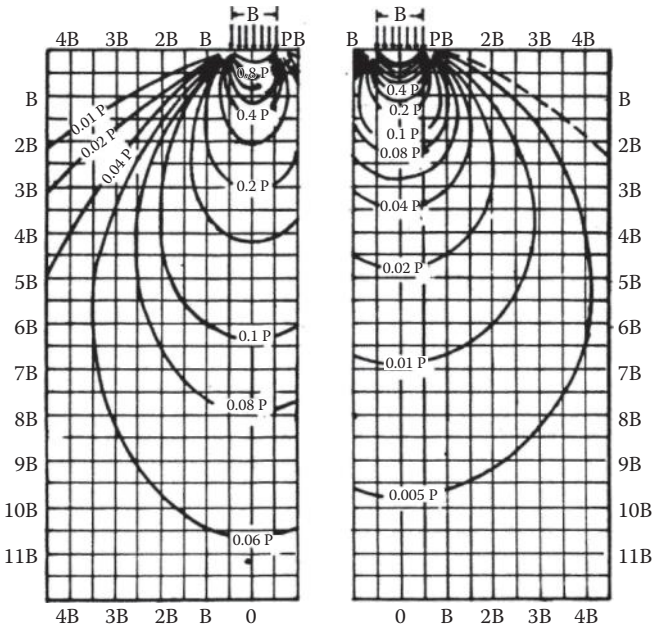
It should be noted that the above methods do not consider the effects of seismic loading on the dynamic properties of the foundation soils. Saturated loose to medium dense cohesionless soils are prone to significant reduction in shear strength commensurate with the increase in the pore pressure due to seismic loading. Reduction in the bearing capacity of spread footing due to such reduction in the soil shear strengths, in particular in case of complete liquefaction, can be more significant than any reduction due to the above inertial effects and thus should be carefully considered. The theoretical static bearing capacity equations may be used with the reduced soil shear strength parameters to estimate seismic bearing capacity. Effects of the load inclination should be considered as above.

8.9 Stress Distributions Beneath Shallow Foundations

Elastic theory is often used to estimate the distribution of stress and settlement as well. Although soils are generally treated as elastic-plastic materials, the use of elastic theory for solving the problems is mainly due to the reasonable match between the boundary conditions for most footings and those of elastic solutions (Holtz 1990). Another reason is due to the lack of availability of acceptable alternatives. Observation and experience have shown that this practice provides satisfactory solutions (Scott 1981; Perloff 1975; Holtz 1990; Bowles 1996).

8.9.1 Semiinfinite, Elastic Foundations

Bossinesq equations based on elastic theory are the most commonly used methods for obtaining subsurface stresses produced by surface loads on semiinfinite, elastic, isotropic, homogeneous, weightless



Square footing
Given
Footing size = 20′ × 20′
Unit pressure P = 2 tsf
Find
Profile of stress increase beneath center of footing due to applied load

B = 20′ P = 2 tsf

Z (ft.)	$\frac{Z}{B}$	σ_z (tsf)
10	0.5	0.70 × 2 = 1.4
20	1	0.38 × 2 = 0.76
30	1.5	0.19 × 2 = 0.38
40	2.0	0.12 × 2 = 0.24
50	2.5	0.07 × 2 = 0.14
60	3.0	0.05 × 2 = 0.10

FIGURE 8.17 Pressure bulbs based on the Bossinesq equation for (a) Long; and (b) square footings. (After Bowles, J. E., *Foundation Analysis and Design*, Fifth Edition, McGraw-Hill Companies, Inc., 1996.)

foundations. Formulas and plots of Bossinesq equations for common design problems are available in NAVFAC (1986). Figure 8.17 shows the isobars of pressure bulbs for square and continuous footings. For other geometry, refer to Poulos and Davis (1974).

8.9.2 Layered Soils

Westergaard (1938), Burmister (1943, 1958, 1967), Sowers and Vesic (1962), Poulos and Davis (1974), and Perloff (1975) discussed the solutions to stress distributions for layered soil strata. The reality of interlayer shear is very complicated due to in situ nonlinearity and material inhomogeneity (Perloff 1975; Holtz 1990). Either zero (frictionless) or with perfect fixity is assumed for the interlayer shear to obtain possible solutions. The Westergaard method assumed that the soil being loaded is constrained by closed spaced horizontal layers that prevent horizontal displacement (NAVFAC, 1986). Figures 8.18 and 8.19 by the Westergaard method can be used for calculating vertical stresses in soils consisting of alternative layers of soft (loose) and stiff (dense) materials.

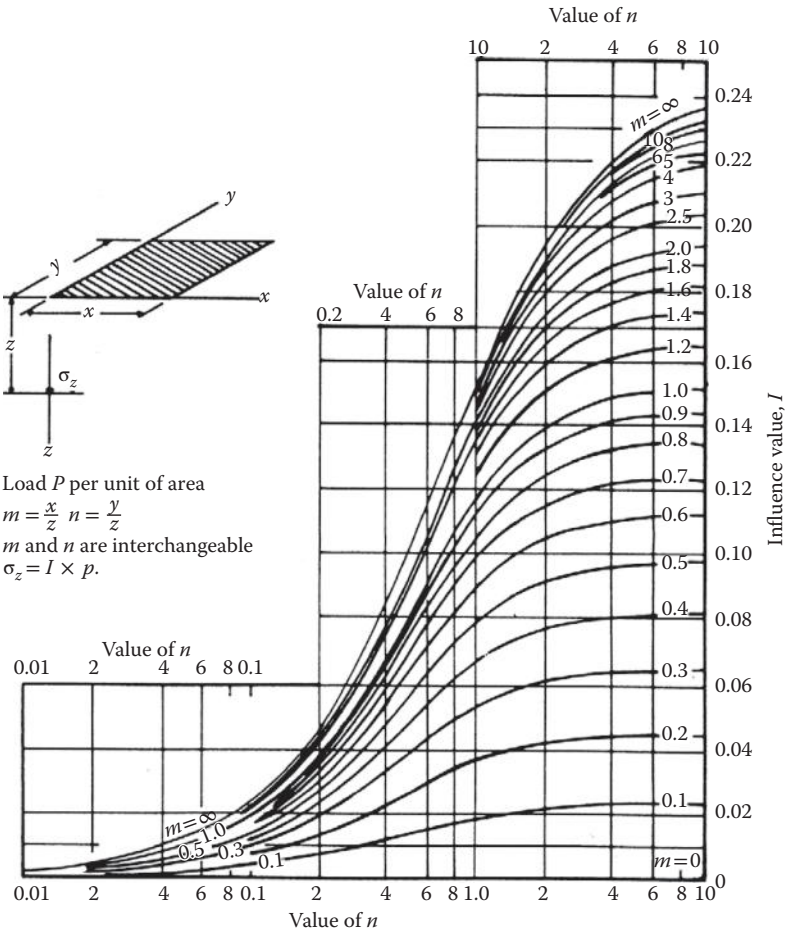


FIGURE 8.18 Influence value for vertical stress beneath a corner of a uniformly loaded rectangular area (Westerqaard case). (After NAVFAC, Design Manual 7.0, Naval Facilities Engineering Command, Department of the Navy, Washington, D.C., 1986.)

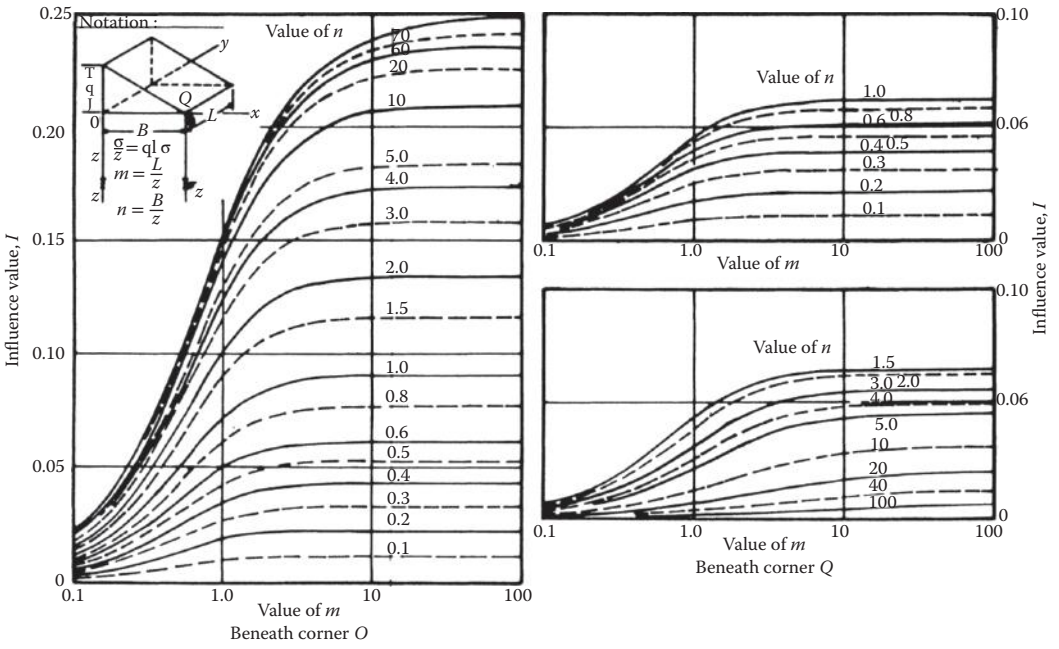
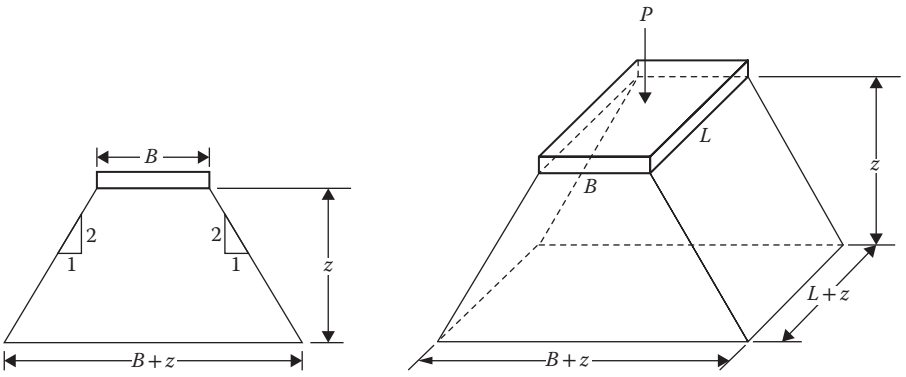


FIGURE 8.19 Influence value for vertical stress beneath triangular load (Westerqaard case). (After NAVFAC, Design Manual 7.0, Naval Facilities Engineering Command, Department of the Navy, Washington, D.C., 1986.)

8.9.3 Simplified Method (2:1 Method)

Assuming a loaded area increasing systematically with depth, a commonly used approach for computing the stress distribution beneath a square or rectangle footing is to use the 2:1 slope method as shown in Figure 8.20. Sometimes a 60° distribution angle (1.73 to 1 slope) may be assumed. The pressure increase Δq at a depth z beneath the loaded area due to base load P is

$$\Delta q = \begin{cases} P/(B'+z)(L'+z) & \text{(for a rectangle footing)} \\ P/(B'+z)^2 & \text{(for a square footing)} \end{cases} \tag{8.34}$$



(a) Strip footing (b) Rectangular footing

FIGURE 8.20 The 2:1 approximation for the distribution of vertical stress with depth for strip and rectangular footings. (From Holtz and Kovacs, *An Introduction to Geotechnical Engineering,* Prentice-Hall, Englewood Cliffs, NJ, 1981.)

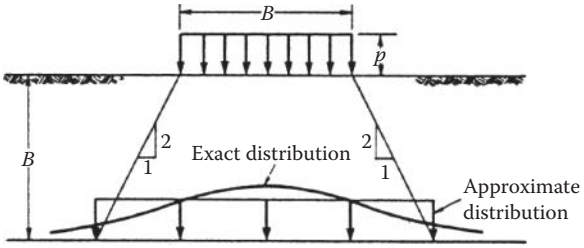


FIGURE 8.21 Relationship between vertical stress below a square uniformly loaded area as determined by approximate and exact methods. (After Perloff, W. H., *Foundation Engineering Handbook*, Second Edition, edited by Fang, H. Y., Chapman & Hall, 1975.)

where symbols are referred to in Figure 8.20. A comparison between the approximate distribution of stress calculated by a theoretical method and the 2:1 method is illustrated in Figure 8.21.

The solutions by this method compare reasonably well with those from theoretical equations from depth $z = B'$ to 4B′ but is not considered accurate for depth z from 0 to B (Bowles 1996). Thus, the use of this method should be limited to preliminary evaluation or design.

8.10 Settlement of Shallow Foundations

The load applied on a footing changes the stress state of the soil below the footing. This stress change may produce a time-dependent accumulation of elastic compression, distortion, or consolidation of the soil beneath the footing. This is often termed the foundation settlement. True elastic deformation consists of a very small portion of the settlement whereas the major components of the settlement are due to a change of void ratio, particle rearrangement, or crushing. Therefore, very little of the settlement will be recovered even if the applied load is removed. The irrecoverable deformation of soil reflects its inherent elastic–plastic stress–strain relationship. The reliability of settlement estimated is influenced principally by soil properties, layering, stress history, and the actual stress profile under the applied load (Bowles 1996; Terzaghi et al. 1996).

In general, the total settlement may be expressed as

$$s_t = s_i + s_c + s_\alpha \tag{8.35}$$

where s_t is the total settlement, s_i the immediate or distortion settlement, s_c the primary consolidation settlement, and s_α is the secondary consolidation settlement. A typical settlement-time history of a shallow foundation is illustrated in Figure 8.22. Although often referred to as elastic settlement, immediate settlement foundation is not elastic in the sense that little or no soil rebound is likely to occur upon unloading. It is referred to as elastic settlement because the elastic theory is often used for computation. The immediate settlement component usually controls in cohesionless soils and very stiff or unsaturated cohesive soils, whereas consolidation settlement usually controls in less-stiff cohesive soils with a degree of saturation above 80% (AASHTO 2002).

8.10.1 Immediate Settlement by Elastic Analysis Methods

On the basis of elastic theory, Steinbrenner (1934) suggested that immediate settlements of footings on sands and clay could be estimated in terms of Young's modulus E of soils. A modified procedure developed by Bowles (1996) may be used for computing settlements (s_i) at the center of flexible footings on the half-space. The settlement equation can be expressed as follows:

$$s_i = q_0' B' (1 - \mu^2) m I_s I_F / E_s \tag{8.36}$$

$$I_s = n[I_1 + \{(1 - 2\mu)/(1 - \mu)\} I_2] \tag{8.37}$$

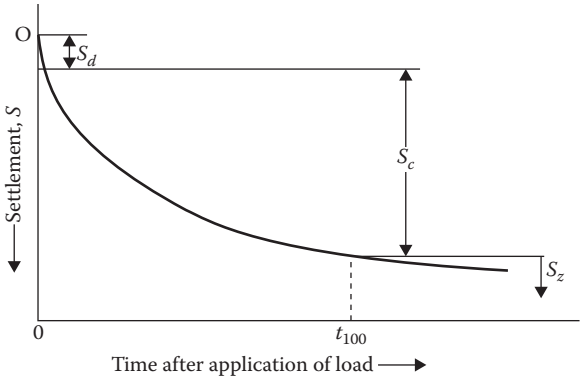


FIGURE 8.22 Schematic time–settlement history of typical point on a foundation. (After Perloff, W. H., *Foundation Engineering Handbook*, Second Edition, edited by Fang, H. Y., Chapman & Hall, 1975.)

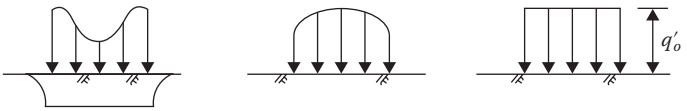


FIGURE 8.23 Approximate distribution of footing contact pressure due to concentrically applied vertical load. (a) contact pressure in cohesive soils; (b) contact pressure in cohesionless soils; and (c) equivalent uniform contact pressure distribution. (After Perloff, W. H., *Foundation Engineering Handbook*, Second Edition, edited by Fang, H. Y., Chapman & Hall, 1975.)

where q'_0 is net increase in the uniform contact pressure, as shown in Figure 8.23, μ and E_s are weighted average values of Poisson's ratio and Young's modulus for compressive soil strata, B' is the least lateral effective (width) dimension the base (convert round bases to equivalent square bases; $B' = 0.5B$ for center and $B' = B$ for corner I_i; $L' = 0.5L$ for center and $L' = L$ for corner I_i), I_i are influence factors depending on dimension of footings, base embedment depth, thickness of soil stratum, and Poisson's ratio (I_1 and I_2 are given in Table 8.13 and I_F is given in Figure 8.24; $M = L'/B'$ and $N = H/B'$), H is the stratum depth causing settlement (see discussion below), m is number of corners contributing to settlement ($m = 4$ at the footing center; $m = 2$ at a side; and $m = 1$ at a corner), and n equals 1.0 for flexible footings and 0.93 for rigid footings.

This equation applies to soil strata consisting of either cohesionless soils of any water content or unsaturated cohesive soils, which may be either organic or inorganic. Highly organic soils (both E_s and μ are subject to significant changes by high organic content) will be dictated by secondary or creep compression rather than immediate settlement; therefore, the applicability of the above equation is limited.

Suggestions were made by Bowles (1996) to appropriately use the equation as follows: (1) make the best estimate of the net increase in contact stress (q'_0) due to the design service load; (2) identify the settlement point to be calculated and divide the base so that the point is at the corner or common corner of one or up to four contributing areas; (3) determine the stratum depth causing settlement that does not approach to infinite rather at either the depth $z = 5B'$ or depth to where a hard stratum is encountered (where E_s in the hard layer is about $10E_s$ of the adjacent upper layer); and (4) calculate the weighted average E_s as following:

$$E_{s,\text{avg}} = \frac{\sum_{n}^{1} H_i E_{si}}{\sum_{n}^{1} H_i} \tag{8.38}$$

TABLE 8.13 Values of I_1 and I_2 to Compute Influence Factors *I*s Used in Equation 8.37

N	*M* = 1.0	1.1	1.2	1.3	1.4	1.5	1.6	1.7	1.8	1.9	2.0
0.2	$I_1 = 0.009$	0.008	0.008	0.008	0.008	0.008	0.007	0.007	0.007	0.007	0.007
	$I_2 = 0.041$	0.042	0.042	0.042	0.420	0.042	0.043	0.043	0.043	0.043	0.043
0.4	0.033	0.032	0.031	0.030	0.029	0.028	0.028	0.027	0.027	0.027	0.027
	0.066	0.068	0.069	0.070	0.070	0.071	0.071	0.072	0.072	0.073	0.073
0.6	0.066	0.064	0.063	0.061	0.060	0.059	0.058	0.057	0.056	0.056	0.055
	0.079	0.081	0.083	0.085	0.087	0.088	0.089	0.090	0.091	0.091	0.092
0.8	0.104	0.102	0.100	0.098	0.096	0.095	0.093	0.092	0.091	0.090	0.089
	0.083	0.087	0.090	0.093	0.095	0.097	0.098	0.100	0.101	0.102	0.103
1.0	0.142	0.140	0.138	0.136	0.134	0.132	0.130	0.129	0.127	0.126	0.125
	0.083	0.088	0.091	0.095	0.098	0.100	0.102	0.104	0.106	0.108	0.109
1.5	0.224	0.224	0.224	0.223	0.222	0.220	0.219	0.217	0.216	0.214	0.213
	0.075	0.080	0.084	0.089	0.093	0.096	0.099	0.102	0.105	0.108	0.110
2.0	0.285	0.288	0.290	0.292	0.292	0.292	0.292	0.292	0.291	0.290	0.289
	0.064	0.069	0.074	0.078	0.083	0.086	0.090	0.094	0.097	0.100	0.102
3.0	0.363	0.372	0.379	0.384	0.389	0.393	0.396	0.398	0.400	0.401	0.402
	0.048	0.052	0.056	0.060	0.064	0.068	0.071	0.075	0.078	0.081	0.084
4.0	0.408	0.421	0.431	0.440	0.448	0.455	0.460	0.465	0.469	0.473	0.476
	0.037	0.041	0.044	0.048	0.051	0.054	0.057	0.060	0.063	0.066	0.069
5.0	0.437	0.452	0.465	0.477	0.487	0.496	0.503	0.510	0.516	0.522	0.526
	0.031	0.034	0.036	0.039	0.042	0.045	0.048	0.050	0.053	0.055	0.058
6.0	0.457	0.474	0.489	0.502	0.514	0.524	0.534	0.542	0.550	0.557	0.563
	0.026	0.028	0.031	0.033	0.036	0.038	0.040	0.043	0.045	0.047	0.050
7.0	0.471	0.490	0.506	0.520	0.533	0.545	0.556	0.566	0.575	0.583	0.590
	0.022	0.024	0.027	0.029	0.031	0.033	0.035	0.037	0.039	0.041	0.043
8.0	0.482	0.502	0.519	0.534	0.549	0.561	0.573	0.584	0.594	0.602	0.611
	0.020	0.022	0.023	0.025	0.027	0.029	0.031	0.033	0.035	0.036	0.038
9.0	0.491	0.511	0.529	0.545	0.560	0.574	0.587	0.598	0.609	0.618	0.627
	0.017	0.019	0.021	0.023	0.024	0.026	0.028	0.029	0.031	0.033	0.034
10.0	0.498	0.519	0.537	0.554	0.570	0.584	0.597	0.610	0.621	0.631	0.641
	0.016	0.017	0.019	0.020	0.022	0.023	0.025	0.027	0.028	0.030	0.031
20.0	0.529	0.553	0.575	0.595	0.614	0.631	0.647	0.662	0.677	0.690	0.702
	0.008	0.009	0.010	0.010	0.011	0.012	0.013	0.013	0.014	0.015	0.016
500.0	0.560	0.587	0.612	0.635	0.656	0.677	0.696	0.714	0.731	0.748	0.763
	0.000	0.000	0.000	0.000	0.000	0.000	0.001	0.001	0.001	0.001	0.001

N	*M* = 2.5	4.0	5.0	6.0	7.0	8.0	9.0	10.0	25.0	50.0	100.0
0.2	$I_1 = 0.007$	0.006	0.006	0.006	0.006	0.006	0.006	0.006	0.006	0.006	0.006
	$I_2 = 0.043$	0.044	0.044	0.044	0.044	0.044	0.044	0.044	0.044	0.044	0.044
0.4	0.026	0.024	0.024	0.024	0.024	0.024	0.024	0.024	0.024	0.024	0.024
	0.074	0.075	0.075	0.075	0.076	0.076	0.076	0.076	0.076	0.076	0.076
0.6	0.053	0.051	0.050	0.050	0.050	0.049	0.049	0.049	0.049	0.049	0.049
	0.094	0.097	0.097	0.098	0.098	0.098	0.098	0.098	0.098	0.098	0.098
0.8	0.086	0.082	0.081	0.080	0.080	0.080	0.079	0.079	0.079	0.079	0.079
	0.107	0.111	0.112	0.113	0.113	0.113	0.113	0.114	0.114	0.114	0.114
1.0	0.121	0.115	0.113	0.112	0.112	0.112	0.111	0.111	0.110	0.110	0.110
	0.114	0.120	0.122	0.123	0.123	0.124	0.124	0.124	0.125	0.125	0.125
1.5	0.207	0.197	0.194	0.192	0.191	0.190	0.190	0.189	0.188	0.188	0.188
	0.118	0.130	0.134	0.136	0.137	0.138	0.138	0.139	0.140	0.140	0.140
2.0	0.284	0.271	0.267	0.264	0.262	0.261	0.260	0.259	0.257	0.256	0.256
	0.114	0.131	0.136	0.139	0.141	0.143	0.144	0.145	0.147	0.147	0.148

(*Continued*)

TABLE 8.13 (*Continued*) Values of I_1 and I_2 to Compute Influence Factors *Is* Used in Equation 8.37

N	*M* = 2.5	4.0	5.0	6.0	7.0	8.0	9.0	10.0	25.0	50.0	100.0
3.0	0.402	0.392	0.386	0.382	0.378	0.376	0.374	0.373	0.368	0.367	0.367
	0.097	0.122	0.131	0.137	0.141	0.144	0.145	0.147	0.152	0.153	0.154
4.0	0.484	0.484	0.479	0.474	0.470	0.466	0.464	0.462	0.453	0.451	0.451
	0.082	0.110	0.121	0.129	0.135	0.139	0.142	0.145	0.154	0.155	0.156
5.0	0.553	0.554	0.552	0.548	0.543	0.540	0.536	0.534	0.522	0.519	0.519
	0.070	0.098	0.111	0.120	0.128	0.133	0.137	0.140	0.154	0.156	0.157
6.0	0.585	0.609	0.610	0.608	0.604	0.601	0.598	0.595	0.579	0.576	0.575
	0.060	0.087	0.101	0.111	0.120	0.126	0.131	0.135	0.153	0.157	0.157
7.0	0.618	0.653	0.658	0.658	0.656	0.653	0.650	0.647	0.628	0.624	0.623
	0.053	0.078	0.092	0.103	0.112	0.119	0.125	0.129	0.152	0.157	0.158
8.0	0.643	0.688	0.697	0.700	0.700	0.698	0.695	0.692	0.672	0.666	0.665
	0.047	0.071	0.084	0.095	0.104	0.112	0.118	0.124	0.151	0.156	0.158
9.0	0.663	0.716	0.730	0.736	0.737	0.736	0.735	0.732	0.710	0.704	0.702
	0.042	0.064	0.077	0.088	0.097	0.105	0.112	0.118	0.149	0.156	0.158
10.0	0.679	0.740	0.758	0.766	0.770	0.770	0.770	0.768	0.745	0.738	0.735
	0.038	0.059	0.071	0.082	0.091	0.099	0.106	0.112	0.147	0.156	0.158
20.0	0.756	0.856	0.896	0.925	0.945	0.959	0.969	0.977	0.982	0.965	0.957
	0.020	0.031	0.039	0.046	0.053	0.059	0.065	0.071	0.124	0.148	0.156
500.0	0.832	0.977	1.046	1.102	1.150	1.191	1.227	1.259	2.532	1.721	1.879
	0.001	0.001	0.002	0.002	0.002	0.002	0.003	0.003	0.008	0.016	0.031

Source: Data from Bowles, J. E., *Foundation Analysis and Design*, Fifth Edition, McGraw-Hill Companies, Inc., 1996.

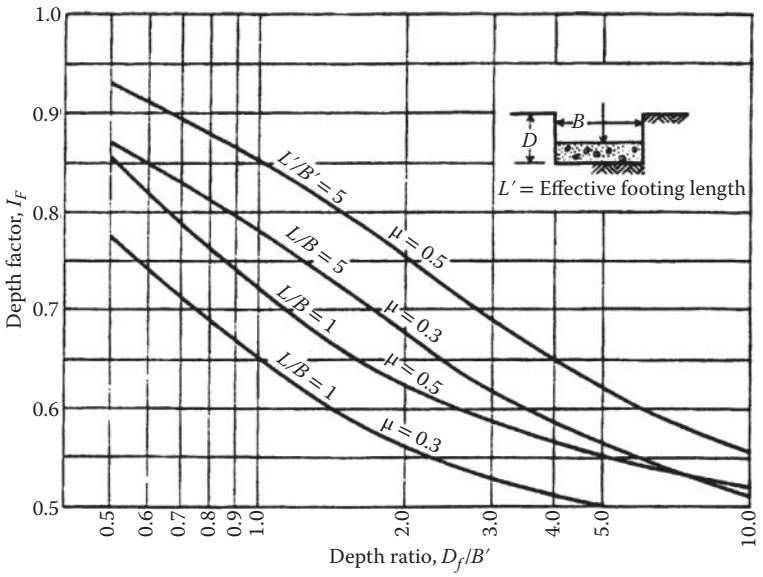


FIGURE 8.24 Influence factor I_F for footing at a depth *D*. (After Bowles, J. E., *Foundation Analysis and Design*, Fifth Edition, McGraw-Hill Companies, Inc., 1996.)

8.10.2 Settlement in Coarse-Grained Soil

Settlement in this type of soils occurs almost instantaneously compared to the rate of application of static service loads—both permanent and live.

8.10.2.1 Elastic Analysis Method

For the normally expected rate of most static loadings, in situ deposits of coarse-grained soils, namely, sand, gravel, and so on, with little or no fine content, in particular clay fraction, can be considered as free draining soils when subjected to additional stresses due to static loading. Most, if not all, of the footing settlement in this type of soil can be expected to occur immediately after the load application. Thus, for all practical purposes, the total footing settlement in this type of soil may be taken as equal to the immediate or elastic settlement evaluated based on Equation 8.36.

That is, for footing founded on free draining cohesionless soils,

$$s_t = s_i \tag{8.39}$$

8.10.2.2 Empirical Methods

The followings empirical methods, in which anticipated total footings settlement in granular soils is correlated with the results of common field tests performed during most routine field exploration, may be used to estimate footing settlement in granular soils.

8.10.2.2.1 SPT Method

D'Appolonia et al. (1970) developed the following equation to estimate settlements of footings on sand using SPT data:

$$s_t = \mu_0 \mu_1 q_0' B'/M \tag{8.40}$$

where μ_o and μ_1 are settlement influence factors that dependent on the footing geometry, depth of embedment, and depth to the relative incompressible layer (Figure 8.25), q_0' is the average net footing base or contact pressures due to the design service load on the foundation, and M is unconfined modulus of soil compressibility. The correlation between M and the average measured SPT blow count (N)

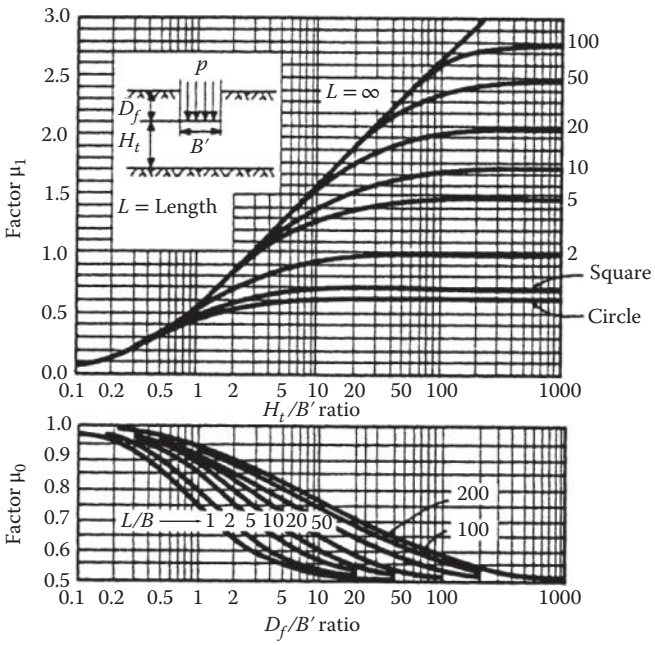


FIGURE 8.25 Settlement influence factors μ_0 and μ_1 for the D'Appolonia et al. procedure (After D'Appolonia, et al., *ASCE Journal of Soil Mechanics and Foundation Division*, **96**, No. SM2, pp. 754–761, 1970.)

corrected to 60% of the theoretical hammer energy, that is, N_{60}, within a depth B′ below footing, is given in Figure 8.26.

Barker et al. (1991) discussed the commonly used procedure for estimating settlement of footing on sand using SPT blow count developed by Terzaghi and Peck (1948, 1967) and Bazaraa (1967).

8.10.2.2.2 CPT Method

Schertmann (1970, 1978) developed a procedure for estimating footing settlements on sand using cone penetration test (CPT) data. This CPT method uses cone-tip penetration resistance, q_c, as a measure of the in situ stiffness (compressibility) soils. Schertmann's method is expressed as following:

$$s_t = C_1 C_2 \Delta p \sum \left(\frac{I_z}{E_s} \right)_i \Delta z_i \tag{8.41}$$

$$C_1 = 1 - 0.5 \left(\frac{\sigma'_{v0}}{\Delta p} \right) \geq 0.5 \tag{8.42}$$

$$C_2 = 1 + 0.2 \log \left(\frac{t_{yr}}{0.1} \right) \tag{8.43}$$

$$E_s = \begin{cases} 2.5 q_c & \text{for square footings (axisymmetric conditions)} \\ 3.5 q_c & \text{for continuous footings with } L'/B' \geq 10 \text{ (plan strain conditions)} \\ \left[2.5 + \left(L'/B' - 1 \right)/9 \right] q_c & \text{for footings with } 1 \leq L'/B' \leq 10 \end{cases} \tag{8.44}$$

where $\Delta p = \sigma'_{vf} - \sigma'_{v0}$ is net contact stress at foundation level, σ'_{v0} is the initial effective in situ overburden stress at the bottom of footings, σ'_{vf} is final effective in situ overburden stress at the bottom of footings, I_z is strain influence factor as defined in Figure 8.27 and Table 8.14, E_s is appropriate Young's modulus at the middle of the ith layer of thickness Δz_I, C_1 is pressure correction factor, C_2 is time rate factor (equal to 1 for immediate settlement calculation or if the lateral pressure is less than the creep pressure determined from PMT), q_c is cone penetration resistance, in pressure units, and Δz is layer thickness.

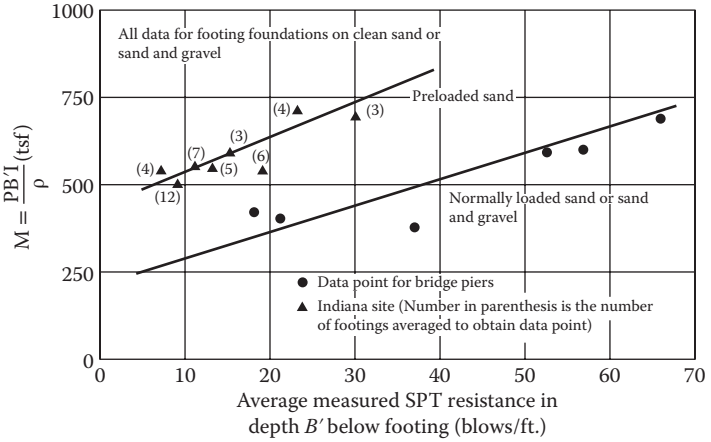


FIGURE 8.26 Correlation between modulus of compressibility and average value standard penetration test (SPT) blow count. (After D'Appolonia et al., *ASCE Journal of Soil Mechanics and Foundation Division*, **96**, No. SM2, pp. 754–761, 1970.)

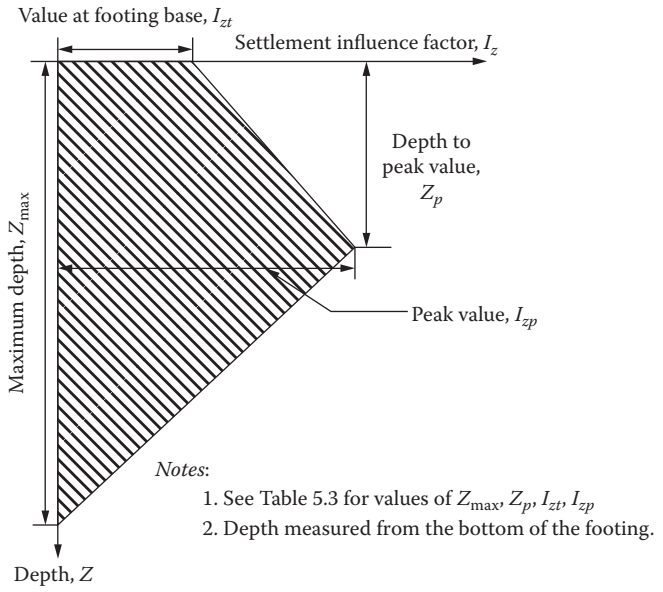

Value at footing base, I_{zt}
Settlement influence factor, I_z
Depth to peak value, Z_p
Maximum depth, Z_{max}
Peak value, I_{zp}
Notes:
1. See Table 5.3 for values of Z_{max}, Z_p, I_{zt}, I_{zp}
2. Depth measured from the bottom of the footing.
Depth, Z

FIGURE 8.27 Variation of Schmertmann's improved settlement influence factors with depth. (After Schertmann, J. H. et al., *ASCE Journal of the Geotechnical Engineering Division*, **104**, No. GT8, pp. 1131–1135, 1978.)

TABLE 8.14 Coefficients to Define the Dimensions of Schmertmann's Improved Settlement Influence Factor Diagram in Figure 31.27

L/B	Maximum Depth of Influence, z_{max}/B'	Depth to Peak Value, z_p/B'	Value of I_z at Top I_{zt}	Peak Value of Stress Influence Factor I_{zp}			
				$\frac{\Delta p}{\sigma'_{vp}} = 1$	$\frac{\Delta p}{\sigma'_{vp}} = 2$	$\frac{\Delta p}{\sigma'_{vp}} = 4$	$\frac{\Delta p}{\sigma'_{vp}} = 10$
1	2.00	0.50	0.10	0.60	0.64	0.70	0.82
2	2.20	0.55	0.11	0.60	0.64	0.70	0.82
4	2.65	0.65	0.13	0.60	0.64	0.70	0.82
8	3.55	0.90	0.18	0.60	0.64	0.70	0.82
≥10	4.00	1.00	0.20	0.60	0.64	0.70	0.82

Source: After Schertmann, J. H. et al., *ASCE Journal of the Geotechnical Engineering Division*, **104**, No. GT8, pp. 1131–1135, 1978.

Note: σ'_{vp} is the initial vertical pressure at the depth of peak influence.

Recent studies by Tan and Duncan (1991) have compared measured settlements with settlements predicted using various procedures for footings on sand. These studies conclude that methods predicting settlements close to the average of measured settlement are likely to underestimate settlements half the time and to overestimate them half the time. The conservative methods (notably Terzaghi and Peck's) tend to overestimate settlements more than half the time and to underestimate them less likely. In other words, there is a trade-off between accuracy and reliability.

A relatively accurate method such as the D'Appolonia et al. (1970) method calculates settlements that are about equal to the average value of actual settlements, but it underestimates settlements half the time (a reliability of 50%). To ensure the calculated settlements equal or exceed the measured settlements about 90% of the time (a reliability of 90%), an adjustment factor of two shall be applied to the settlements predicted by the D'Appolonia et al. method. Table 8.15 shows values of adjustment factor for 50% and 90% reliability in settlement predicted using Terzaghi and Peck (1967), D'Appolonia et al. (1970), and Schertmann (1978) methods.

TABLE 8.15 Value of Adjustment Factor for 50% and 90% Reliability in Displacement Estimates

		Adjustment Factor	
Method	Soil Type	For 50% Reliability	For 90% Reliability
Terzaghi and Peck	Sand	0.45	1.05
Schmertmann	Sand	0.60	1.25
D'Appolonia et al.	Sand	1.00	2.00

8.10.3 Settlement in Fine-Grained Soils

For footing in clay and other fine-grained soils with relatively low permeability compared to the rate of static load application, the total settlement (s_t) is the sum of the immediate or elastic settlement evaluated based on Equation 8.36 and the consolidation settlements as discussed in Section 8.10.3.1.

Elastic settlement occurs almost instantaneously compared to the rate of application of static service loads—both permanent and live. Thus, similar to footings in coarse-grained soils, both permanent and live service loads need to be included in estimating the immediate settlement of footing in fine-grained soils.

8.10.3.1 Consolidation Settlement

The majority of the settlement in saturated fine-grained soils, in particular in very soft to medium stiff clays, is time dependent. For computational purposes, this time-dependent settlement can be divided into two components: primary consolidation settlement (s_c) and secondary settlement (s_α).

The primary consolidation settlement (s_c) occurs due to the slow expulsion of the pore water and hence the dissipation of the excess pore water pressure generated by the application of the static service loads at the rates normally expected of permanent loads. As the excess pore water pressure dissipates, the net effective vertical stress on the soil increases as the primary consolidation continues to occur.

The secondary consolidation settlement (s_c) occurs after the completion of the primary settlement (i.e., after the complete dissipation of the generated pore water pressure) due mainly to the reorientation of the soil particles under essentially constant effective vertical stress.

Little or no consolidation settlement occurs in fine-grained soils due to static transient live loads. Thus, only the permanent loads need to be considered in evaluating the consolidation settlement.

8.10.3.1.1 Primary Consolidation Settlement (s_c)

The total amount of settlement due to primary consolidation can be estimated using Terzaghi's one-dimensional consolidation theory (Terzaghi 1943; Lambe and Whitman 1969; Peck et al. 1974; Terzaghi et al. 1996) as follows:

$$s_c = \begin{cases} \dfrac{H_c}{1+e_0}\left[C_r \log\left(\dfrac{\sigma'_p}{\sigma'_{v0}}\right) + C_c \log\left(\dfrac{\sigma'_{vf}}{\sigma'_p}\right)\right] & (\text{for OC soils, i.e., } \sigma'_p > \sigma'_{v0}) \\ \dfrac{H_c}{1+e_0}\left[C_c \log\left(\dfrac{\sigma'_{vf}}{\sigma'_p}\right)\right] & (\text{for NC soils, i.e., } \sigma'_p = \sigma'_{v0}) \end{cases} \tag{8.45}$$

where H_c is the height of compressible layer, e_0 is void ratio at initial vertical effective stress, C_c is the compression index (see Table 8.16), C_r is recompression index (also see Table 8.16), σ'_p is

TABLE 8.16 Some Empirical Equations for C_c and C_r

Compression Index	Source	Comment
$C_c = 0.009(\text{LL} - 10)$	Terzaghi and Peck (1967)	$S_t \leq 4$, $\text{LL} < 100$
$C_c = 0.2343e_0$	Nagaraj and Srinivasa Murthy (1986)	
$C_c = 0.5G_s(PI/100)$	Worth and Wood (1978)	Modified Cam Clay model
$C_c = PI/74$	EPRI (1990)	
$C_c = 0.37(e_0 + 0.003w_L + 0.0004w_N - 0.34)$	Azzouz et al. (1976)	Statistical analysis
Recompression Index	Source	
$C_r = 0.0463w_L G_s$	Nagaraj and Srinivasa Murthy (1986)	

Note: S_t = (Peak undrained shear strength/Undrained residual shear strength); LL= Liquid Limit (%), PI= Plasticity Index (%), eo = Initial in-situ void ratio, Gs=Soil specific gravity, and w_L= Liquid Limit (in decimal) and W_N= In-situ natural water content (in decimal).

TABLE 8.17 Secondary Compression Index

C_α/C_c	Material
0.02 ± 0.01	Granular soils including rockfill
0.03 ± 0.01	Shale and mudstone
0.04 ± 0.01	Inorganic clays and silts
0.05 ± 0.01	Organic clays and silts
0.06 ± 0.01	Peat and muskeg

Source: Data from Terzaghi, K. et al., *Soil Mechanics in Engineering Practice*, Third Edition, John Wiley & Sons, Inc., 1996.

maximum past vertical effective stress, σ'_{v0} is initial vertical effective stress, σ'_{vf} is final vertical effective stress. Highly compressible cohesive soils are rarely chosen to place footings for bridges where tolerable amount of settlement is relative small. Preloading or surcharging to produce more rapid consolidation has been extensively used for foundations on compressible soils (Perloff 1975). Alternative foundation systems would be appropriate if large consolidation settlements are expected to occur.

8.10.3.1.2 Secondary Consolidation Settlement

Settlements of footings on cohesive soils continuing beyond primary consolidation are called secondary settlement. Secondary settlement develops at a slower and continually decreasing rate, and may be estimated as follows:

$$s_\alpha = C_\alpha H_t \log \frac{t_{sc}}{t_p} \tag{8.46}$$

where C_α is coefficient of secondary settlement, and normally given as ratio to C_c (see Table 8.17), H_t is total thickness of layers undergoing secondary settlement, t_{sc} is time for which secondary settlement is calculated and t_p is time, in the same unit as t_{sc}, to the end of primary consolidation settlement.

8.10.4 Tolerable Settlement

Criteria for tolerable foundation settlement shall be established consistent with the function and type of the bridge structure, anticipated service life, and consequences of unacceptable movements on structure performance as outlined by AASHTO (2012). As discussed earlier, the criterion adopted by AASHTO (2012) considering the angular distortion (δ/l) between adjacent footings is as follows:

$$\frac{\delta}{l} \leq \begin{cases} 0.008 & \text{for simple span bridge} \\ 0.004 & \text{for continuous span bridge} \end{cases} \tag{8.47}$$

where δ is differential settlement of adjacent support and l is center-center spacing between adjacent supports.

8.11 Shallow Foundations on Rock

Wyllie (1992) outlines the following examinations that are necessary for designing shallow foundations on rock: (1) the bearing capacity of the rock to ensure that there will be no crushing or creep of the material within the loaded zone; (2) settlement of the foundation that will result from elastic strain of the rock and possibly inelastic compression of weak seams within the volume of rock compressed by the applied load; (3) sliding and shear failure of blocks of rock formed by intersecting fractures within the foundation. This condition usually occurs where the foundation is located on a steep slope and the orientation of the fractures is such that the blocks can slide out of the free face.

USACE (1994) provides detailed guidelines for the characterization of rock as foundation support material as well as the methodologies for the evaluation of settlement and ultimate bearing capacity or nominal bearing resistance in compression of spread footings founded on rock.

8.11.1 Presumptive Allowable Bearing Pressures

It is common to use allowable bearing capacity for various rock types listed in building codes for footing design. As provided in Table 8.18, presumptive allowable bearing pressures have been developed to limit settlement to within permissible amount.

8.11.2 Allowable Bearing Pressures/Ultimate Bearing Capacity of Fractured Rock

Various empirical procedures for estimating allowable bearing pressure of foundations on fractured rock are available in literature. Peck et al. (1974) suggested an empirical procedure for estimating allowable bearing pressures of foundations on jointed rock based on rock quality designation (RQD) index. The predicted allowable bearing pressure by this method should be used with the assumption that the spread footing foundation may experience settlement up to about 12. 7 mm (0.5 in.) (Peck et al. 1974).

Carter and Kulhawy (1988) proposed an empirical approach for estimating ultimate bearing capacity of fractured rock. Their method is based on unconfined compressive strength of the intact rock core sample and rock mass quality. Wyllie (1992) detailed an analytical procedure for computing bearing capacity of fractured rock mass using Hoek-Brown strength criterion. Details of rational methods for the topic can also be found in Kulhawy and Goodman (1987), Goodman (1989).

TABLE 8.18 Presumptive Allowable Bearing Pressures (tsf) for Spread Footing Foundations on Rock

Code	Year[a]	Bedrock[b]	Sound Foliated Rock	Sound Sedimentary Rock	Soft Rock[c]	Soft Shale	Broken Shale
Baltimore	1962	100	35		10		
BOCA	1970	100	40	25	10	4	
Boston	1970	100	50	10	10		1.5
Chicago	1970	100	100				
Cleveland	1951/1969			25			
Dallas	1968	$0.2q_u$	$2q_u$	$0.2q_u$	$0.2q_u$	$0.2q_u$	$0.2q_u$
Detroit	1956	100	100	9600	12	12	
Indiana	1967	$0.2q_u$	$2q_u$	$0.2q_u$	$0.2q_u$	$0.2q_u$	$0.2q_u$
Kansas	1961/1969	$0.2q_u$	$2q_u$	$0.2q_u$	$0.2q_u$	$0.2q_u$	$0.2q_u$
Los Angeles	1970	10	4	3	1	1	1
New York City	1970	60	60	60	8		
New York State		100	40	15			
Ohio	1970	100	40	15	10	4	
Philadelphia	1969	50	15	10–15	8		
Pittsburgh	1959/1969	25	25	25	8	8	
Richmond	1968	100	40	25	10	4	1.5
St. Louis	1960/1970	100	40	25	10	1.5	1.5
San Francisco	1969	3–5	3–5	3–5			
UBC	1970	$0.2q_u$	$2q_u$	$0.2q_u$	$0.2q_u$	$0.2q_u$	$0.2q_u$
NBC Canada	1970			100			
New South Wales, Australia	1974			33	13	4.5	

Source: After Putnam, J. B. Analysis and Design of Foundations on Continuous Rock, M. S. Thesis, Syracuse University, May, Syracuse, New York, 1981.

Note: q_u = unconfined compressive strength.

[a] Year of code or original year and date of revision.

[b] Massive crystalline bedrock.

[c] Soft and broken rock, not including shale.

[d] Allowable bearing pressure to be determined by appropriate city official.

8.11.3 Settlements of Foundations on Rock

Wyllie (1992) summarizes the settlements of foundations on rock as following three different types:

1. Elastic settlements result from a combination of strain of the intact rock, slight closure and movement of fractures, and compression of any minor clay seams (less than a few millimeters). Elastic theory can be used to calculate this type of settlement. Detail information can be found in Wyllie (1992), Kulhaway (1978), USACE (1994) and AASHTO (2002).
2. Settlements result from the movement of blocks of rock due to shearing of fracture surfaces. This occurs when foundations are sitting at the top of a steep slope and unstable blocks of rock are formed in the face. The stability of foundations on rock is influenced by the geological characterization of rock blocks. The information required on structural geology consists of the orientation, length and spacing of fractures, and their surface and infilling materials. Procedures have been developed for identifying and analyzing the stability of sliding blocks (Wyllie 1992), stability of wedge blocks (Hoek and Bray 1981), stability of toppling blocks (Goodman and Bray 1976), or three-dimensional stability of rock blocks (Goodman and Shi 1985).

3. Time-dependent settlement occurs when foundations found on the rock mass which consisting of substantial seams of clay or other compressible materials. This type of settlements can be estimated using the procedures described in Section 8.11. Also the time-dependent settlement can occur if foundations found on ductile rocks such as salt where strains develop continuously at any stress level, or brittle rocks when the applied stress exceeds the yield stress.

8.12 Structural Design of Shallow Foundations

The plan dimensions of a spread footing (B and L) are controlled by the nominal bearing resistance of the soil and stiffness of the soil to prevent damages to the superstructure caused by support movements. Maximum contact bearing stress under LRFD strength and extreme event load combinations shall be less than factored nominal bearing resistance of the soil. Furthermore, settlement and rotation of the footing under LRFD service limit state load combinations shall be within acceptable limits. Such limits depend on the continuity of the superstructure, type of the structural material and system, span length, variations in the geotechnical properties from one support to the next, and number of columns per support. Considering complexity of settlement analysis and variables involved, some design codes have specified conservative limits for acceptable support settlement. Rotation of the support is controlled by specifying upper limits on eccentricity of the loads under service and extreme event loads applied to the footing. Such limits depend on stiffness and shear strength of the soil and will be different for soil and rock.

The bearing stress distribution beneath the footing depends on rigidity of the footing, type of the soil, soil stress–strain relationship, and time-dependent response to contact stresses. Common types of distribution are uniform (footings on soil) and linear (footings on rock) as shown in Figure 8.28a and b, respectively. Irrespective of the stiffness of the soil, linear distribution is assumed for structural analysis (concrete and steel design) of the footing.

The depth of the spread footing (D) must be adequate to provide enough resistance against one-way (direct) and two-way (punching) shears on surfaces shown in Figure 8.29a and b; allow development of the column rebar into the footing; and provide enough flexural and shear resistance against stresses

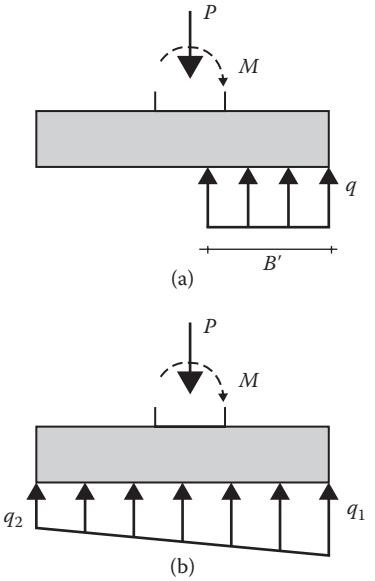


FIGURE 8.28 (a) Distribution of bearing stress for spread footings on soil. (b) Distribution of bearing stress for spread footings on rock.

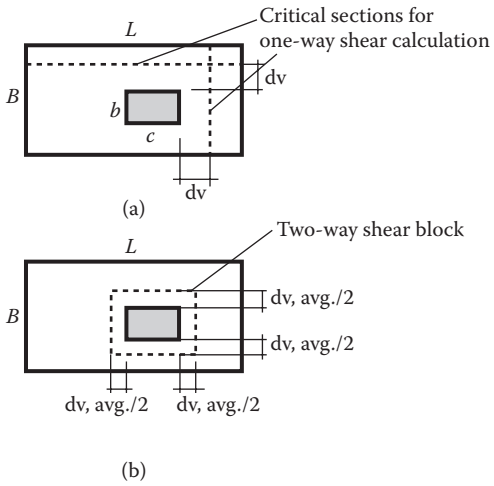


FIGURE 8.29 (a) Critical section for one-way shear. (b) Critical section for two-way shear.

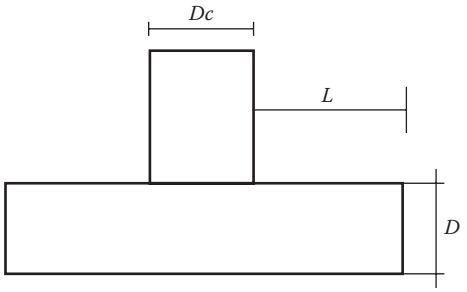


FIGURE 8.30 Limitations on the depth of the footing.

applied by the soil. The one-way shear action may control the depth for rectangular footings if the L/B ratio is greater than about 1.2 and may control for other L/B ratios when there is overturning or eccentric loading.

Studies by Duan and McBride (1995) indicated that if the ratio of cantilevered length to the depth of the spread footing or pile cap (L/D shown in Figure 8.30) is greater than 2.5, nonlinear bearing stress distribution must be assumed and hand calculations will not be accurate. However Caltrans' Seismic Design Criteria (2011b) reduces this ratio to $L/2.2$ for design purposes. Furthermore, if seismic forces are considered in design, depth of the footing (D) is recommended to be at least 0.8 of the diameter of the column (D_c) that it supports.

Sliding of the footing must be checked for strength and extreme event limit states. Resistance against sliding is provided by friction between the footing and the soil, as well as passive resistance of the backfill soil against the face of the footing. Resistance factors may be assumed 1.0 for extreme event combinations.

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References

AASHTO. 2002. *Standard Specifications for Highway Bridges*, Seventeenth Edition, American Association of State Highway and Transportation Officials, Washington, DC.

AASHTO. 2012. *AASHTO LRFD Bridge Design Specifications*, Customary US Units, 2012, American Association of State Highway and Transportation Officials, Washington DC.

Awkati 1970. Unpublished work as cited in Schmertmann (1978)

Azzouz, A. S., Krizek, R. J., and Corotis, R. B. 1976. Regression analysis of soil compressibility, *JSSMFE Soils and Foundations*, 16(2), 19–29.

Baguelin, F., Jezequel, J. F., and Shields, D. H. 1978. *The Pressuremeter and Foundation Engineering*, Transportation Technical Publications, Clausthal, Germany, 617 pp.

Barker, R. M., Duncan, J. M., Rojiani, K. B., Ooi, P. S. K., Tan, C. K., and Kim, S. G. 1991. *Manuals for the Design of Bridge Foundations*, National Cooperative Highway Research Program Report 343, Transportation Research Board, National Research Council, Washington, DC.

Bazaraa, A. 1967. *Use of Standard Penetration Test for Estimating Settlements of Shallow Foundations on Sands*, Ph.D. Dissertation Submitted to Department of Civil Engineering, University of Illinois, Urbana, Illinois, 380 pp.

Bowles, J. E. 1975. Spread footings, Chapter 15 in *Foundation Engineering Handbook*, Edited by Winterkorn, H. F., and Fang, H. Y. Van Nostrand Reinhold, New York, NY.

Bowles, J. E. 1982. *Foundation Analysis and Design*, Third Edition, McGraw-Hill, New York, NY.

Bowles, J. E. 1988. *Foundation Analysis and Design*, Fourth Edition, McGraw-Hill, New York, NY.

Bowles, J. E. 1989. *Foundation Analysis and Design*, Fourth Edition, McGraw-Hill, New York, NY.

Bowles, J. E. 1995. *Foundation Analysis and Design*, Fifth Edition, McGraw-Hill, New York, NY.

Bowles, J. E. 1996. *Foundation Analysis and Design*, Fifth Edition, McGraw-Hill, New York, NY.

Briaud, J. L. 1986. Pressuremeter and foundation design, *Proceedings of the Conference on Use of in situ Tests in Geotechnical Engineering*, ASCE Geotechnical Publication No. 6, pp. 74–116.

Briaud, J. L. 1992. *The Pressuremeter*, A. A. Balkema Publishers, Brookfield, Blacksburg, Virginia, VT.

Burmister, D. M. 1943. The theory of stresses and displacements in layered systems and application to the design of airport runways, *Proceedings of the Highway Research Board*, **23**, 126–148.

Burmister, D. M. 1958. Evaluation of pavement systems of WASHO road test layered system methods, *Highway Research Board Bulletin*, No. 177.

Burmister, D. M. 1967. Applications of dimensional analyses in the evaluation of asphalt pavement performances, Paper for presentation at Fifth Paving Conference, Albuquerque, NM.

Budhu, M, and Al-Karni, A. A. 1993. Seismic bearing capacity of soils, *Geotechnique*, 43(1), 181–187.

Caltrans. 2008. *Memo to Designers (MTD) 4-1, Spread Footing*, California Department of Transportation, Sacramento, CA.

Caltrans. 2011a, November. *California Amendments to AASHTO Bridge Design Specifications*, Fourth Edition, California Department of Transportation, Sacramento, CA.

Caltrans. 2011b. *Seismic Design Criteria*, California Department of Transportation, Sacramento, CA.

Canadian Geotechnical Society. 1985. *Canadian Foundation Engineering Manual*, Second Edition, Canadian Geotechnical Society, Richmond, BC, Canada, 456 pp.

Caquot, A., and Kerisel, F. 1948. *Tables for the Calculations of Passive Pressure, Active Pressure and Bearing Capacities of Foundations*, Gauthier-Villars, Paris.

Carter, J. P., and Kulhawy, F. H. 1988. *Analysis and Design of Drilled Shaft Foundations Socketed into Rock*, Report No. EL-5918, Empire State Electric Engineering Research Corporation and Electric Power Research Institute, New York, NY.

Chen, W. F. 1975. *Limit Analysis and Soil Plasticity*, Elsevier, Amsterdam, The Netherlands.

Choudhury, D. and Rao, K. S. 2006. Seismic bearing capacity of shallow strip foundations embedded in slope, *ASCE International Journal of Geomechanics*, 6(3), 176–184.

D'Appolonia, D. J., D'Appolonia, E., and Brisette, R. F. 1970. Settlement of spread footings on sand (closure), *ASCE Journal of Soil Mechanics and Foundation Division*, 96(2), 754–761.

Dormieux, L., and Pecker, A. 1995. Seismic bearing capacity of foundations on cohesionless soils, *ASCE Journal of Geotechnical Engineering*, 123(3), 300–303.

Duan, L., and McBride, S. B. 1995. The effects of cap stiffness on pile reactions, *Concrete International*, American Concrete Institute 17(1), 42–44.

FHWA. 2002. *Evaluation of Soil and Rock Properties, Geotechnical Circular Number 5*, Federal Highway Administration, Washington DC.

Gifford, D. G., Kraemen, S. R., Wheeler, J. R., and McKnown, A. F. 1987. *Spread Footing for Highway Bridges*, Report No. RD-86/185, Federal Highway Administration, Washington, DC, 229 pp.

Goodman, R. E. 1989. *Introduction to Rock Mechanics*, Second Edition, John Wiley & Sons, New York, NY.

Goodman, R. E., and Bray, J. W. 1976. Toppling of rock slopes, *Proceedings of the Specialty Conference on Rock Engineering for Foundations and Slopes*, ASCE, **2**, pp. 201–234, Boulder, CO.

Goodman, R. E., and Shi, G. 1985. *Block Theory and its Application to Rock Engineering*, Prentice-Hall, Englewood Cliffs, NJ.

Hansen, B. J. 1970. *A Revised and Extended Formula for Bearing Capacity*, Bulletin No. 28, Danish Geotechnical Institute, Copenhagen, Denmark, pp. 5–11.

Hoek, E., and Bray, J. 1981. *Rock Slope Engineering*, Second Edition, IMM, London.

Holtz, R. D. 1990. Stress distribution and settlement of shallow foundations, Chapter 5 in *Foundation Engineering Handbook*, Second Edition, Edited by Fang, H. Y. Chapman & Hall, New York, NY.

Hunt, R. E. 1986. *Geotechnical Engineering Analysis Evaluation*, McGraw-Hill, New York, NY.

Idriss, I. M., and Boulanger, W. W. 2008. *Soil Liquefaction During Earthquakes*, EERI Monograph No. MNO-12, Oakland, CA.

Karol, R. H. 1960. *Soils and Soil Engineering*, Prentice-Hall, Englewood Cliffs, NJ.

Kulhawy, F. H., and Goodman, R. E. 1987. Foundation in rock, Chapter 55 in *Ground Engineering Reference Manual*, Edited by F. G. Bell, Butterworths, London.

Kulhawy, F. H., and Mayne, P. W. 1990. *Manual on Estimating Soil Properties for Foundation Design*, Electric Power Research Institute, EPRI EL-6800, Project 1493-6, Final Report, August, 1990.

Kulhaway, F. H. 1978. Geomechanical Model for Rock Foundation Settlement, *Journal of Geotechnical Engineering*, ASCE, 104(2), 211–227.

Kutter, B. L., Abghari, A., and Cheney, J. A. 1988. "Strength parameters for bearing capacity in sand", *ASCE Journal of Geotechnical Engineering Division,* 114(4), 491–498.

Kumar, J, and Rao, V.B.K.M. 2003. Seismic bearing capacity factors for spread foundations, *Geotechnique,* 52(2), 79–88.

Lambe, T. W., and Whitman, R. V. 1969. *Soil Mechanics*, John Wiley & Sons, New York, NY.

Menard, L. 1965. Regle pour le calcul de la force portante et du tassement des fondations en fonction des resultats pressiometriques, *Proceedings of the Sixth International Conference on Soil Mechanics and Foundation Engineering*, Montreal, **2**, pp. 295–299.

Meyerhof, G. G. 1951. The ultimate bearing capacity of foundations, *Geotechnique*, 2(4), 301–331.

Meyerhof, G. G. 1956. Penetration tests and bearing capacity of cohesionless soils, *ASCE Journal of Soil Mechanics and Foundation Division*, 82(1), 1–19.

Meyerhof, G. G. 1957. The Ultimate Bearing Capacity of Foundations on Slopes, *Proceedings of the fourth International Conference on Soil Mechanics and Foundation Engineering*, London.

Meyerhof, G. G. 1963. Some recent research on the bearing capacity of foundations, *Canadian Geotechnical Journal*, 1(1), 16–26.

Moulton, L. K., Ganga Rao, H. V. S., and Halvorsen, G. T. 1985. Tolerable Movement Criteria for Highway Bridges Final Report, Report No. FHWA/RD-85/107, October 1985. (NTIS PB86-150745/AS).

Munfakh, G., Arman, A., Collin, J. G., Hung, J. C., and Brouillette, R. P. 2001. *Shallow Foundations*, No. NHI-01-023, Federal Highway Administration, Washington, DC.

Nagaraj, T. S., and Srinivasa Murthy, B. R. 1986. A critical reappraisal of compression index, *Geotechnique*, 36(1), 27–32.

NAVFAC. 1986. *Foundations and Earth Structures, DM 7.02*, Naval Facilities Engineering Command, Alexandria, VA.

NAVFAC Design Manual 7.01. 1986. *Soil Mechanics*, Naval Facilities Engineering Command, Department of the Navy, Washington, DC.

NCHRP. 1991. *Manual for the Design of Bridge Foundations*, National Cooperative Highway Research Program Report No. 343, National Science Foundation, Washington, DC.

O'Neill, M. W., Townsend, F. C., Hassan, K. H., Buller, A., and Chan, P. S. 1996. *Load Transfer for Drilled Shafts in Intermediate Geomaterials*, Publication No. FHWA-RD-95-171, Federal Highway Administration, McClean, VA.

Olsen, S. M., and Johnson, C. I. 2008. Analyzing liquefaction-induced laterals spreads using strength ratio, *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, 134(8), 1035–1049.

Paloucci, R., and Pecker, A. 1997. Seismic bearing capacity of shallow foundations on dry soils, soils and foundations, *Journal of the Japanese Geotechnical Society,* 37(3), 95–105.

Peck, R. B., Hanson, W. E., and Thornburn, T. H. 1974. *Foundation Engineering*, Second Edition, John Wiley & Sons, New York, NY.

Perloff, W. H. 1975. Pressure distribution and settlement, Chapter 4 in *Foundation Engineering Handbook*, Second Edition, Edited by Fang, H. Y. Chapman & Hall, New York, NY.

Poulos, H. G., and Davis, E. H. 1974. *Elastic Solutions for Soil and Rock Mechanics*, John Wiley & Sons, New York, NY.

Prandtl, L. 1920. *Uber die Harte Platischer korper*, Nach. Geseel. Wiss, Gottingen, Math-Phys., Kl,., pp. 75–85.

Prandtl, L. 1921. *"Über die Eindringungsfestigkeit (Härte) plastischer Baustoffe und die Festigkeit von Schneiden" (On the penetrating strengths (hardness) of plastic construction materials and the strength of cutting edges),* Zeit. Angew. Math. Mech., 1(1), 15–20.

Putnam, J. B. 1981. Analysis and Design of Foundations on Continuous Rock, M. S. Thesis, Syracuse University, May, Syracuse, New York.

Randolph, M. F. 2003. Science and empiricism in pile foundation design, *Geotechnique*, 53(10), **53**, 847–875.

Reissner, H. 1924. Zum Erddruckprolem, *Proceedings of the first International Congress of Applied Mechanics*, Delft, The Netherlands, pp. 295–311.

Richards, R., Elms, D.G., and Budhu, M. 1993. Seismic bearing capacity and settlement of foundations, *Journal of Geotechnical Engineering,* 119(4), 662–674.

Robertson, P.K. 2013. Liquefaction and liquefied strength using the cone penetration test, *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, 136(6), 842–853.

Seed, R. B., and Harder, L. F. 1990. SPT-based analysis of cycle pore pressure generation and undrained residual shear strength, Proceedings, Seed Memorial Symposium, Edited by J. M. Duncan, BiTech Publishers, Vancouver, BC, Canada, pp. 351–376.

Schertmann, J. H. 1970. Static cone to compute static settlement over sand, *ASCE Journal of the Soil Mechanicals and Foundation Division*, 96(3), 1011–1043.

Schertmann, J. H. 1978. *Guidelines for Cone Penetration Test Performance, and Design*, Federal Highway Administration, Report FHWA-TS-78-209.

Schertmann, J. H., Hartman, J. P., and Brown, P. R. 1978. Improved strain influence factor diagrams, *ASCE Journal of the Geotechnical Engineering Division*, 104(8), 1131–1135.

Scott, R. F. 1981. *Foundation Analysis*, Prentice-Hall, Englewood Cliffs, NY.

Scott, R. F., and Schoustra, J. J. 1968. *Soil Mechanics and Engineering*, McGraw-Hill, New York, NY.

Skempton, A. W., and MacDonald, D. H. 1956. The allowable settlement of buildings, *Proceedings of the Institution of Civil Engineers*, London, 5, Part III, pp. 727–768.

Skempton, A. W. 1986. Standard penetration test procedures and the effects in sands of overburden pressure, relative density, particle size, aging, and overconsolidation, *Geotechnique,* 36(3), 425–447.

Sowers, G. F., and Vesic, A. B. 1962. Vertical stresses in subgrades beneath statically loaded flexible pavements, *Highway Research Board Bulletin*, No. 342.

Stark, T. D., and Mesri, G. 1992. Undrained shear strength of sands for stability analysis, *ASCE Journal of the Geotechnical Engineering*, 118(11), 1727–1747.

Steinbrenner, W. 1934. *Tafeln zur Setzungberechnung*, Die Strasse, pp. 121–124.

Tan, C. K., and Duncan, J. M. 1991. Settlement of footings on sand – accuracy and reliability, *Proceedings of Geotechnical Congress*, Boulder, CO.

Terzaghi, K. 1943. *Theoretical Soil Mechanics*, John Wiley & Sons, New York, NY.

Terzaghi, K., and Peck, R. B. 1948. *Soil Mechanics in Engineering Practice*, First Edition, John Wiley & Sons, New York, NY.

Terzaghi, K., and Peck, R. B. 1967. *Soil Mechanics in Engineering Practice*, Second Edition, John Wiley & Sons, New York, NY.

Terzaghi, K., and Peck, R. B., Mesri, G. 1996. *Soil Mechanics in Engineering Practice*, Third Edition, John Wiley & Sons, New York, NY.

Tomlinson, M. J., and Boorman, R. 2001. *Foundation Design and Construction*. Seventh Edition, Prentice Hall, Upper Saddle River, NJ.

TRCC. 2009. *Limited Statutory Warranty and Building and Performance Standards*, Effective June 1, 2005, as amended through July 31, 2009, Texas Property Code Title 16 and Texas Administrative Code 10, Chapter 304, Texas Residential Construction Commission.

Tschebotarioff, G. 1951. *Soil Mechanics, Foundations and Earth Structures,* McGraw-Hill, New York.

USACE. 1995. Rock Foundations, EM 1110-1-2908, U.S. Army Corps of Engineers, Washington, DC.

USACE. 2005. *Stability Analyses of Hydraulic Structures*, EM 1110-2-2100, U.S. Army Corps of Engineers.

Vesic, A. S. 1963. Bearing capacity of deep foundations in sand, National Academy of Sciences, *National Research Council, Highway Research Record*, 39, 112–153.

Vesic, A. S. 1973. Analysis of ultimate loads of shallow foundations, *ASCE Journal of the Soil Mechanics and Foundation Engineering Division*, 99(1), 45–73.

Vesic, A. S. 1975. Bearing capacity of shallow foundations, Chapter 3 in *Foundation Engineering Handbook*, Edited by Winterkorn, H. F., and Fang, H. Y. Van Nostrand Reinhold, New York, NY.

Westergaard, H. M. 1938. A Problem of Elasticity Suggested by a Problem in Soil Mechanics: Soft Material Reinforced by Numerous Strong Horizontal Sheets, in *Contributions to the Mechanics of Solids*, Stephen Timoshenko Sixtieth Anniversary Volume, Macmillan, New York, NY.

Wroth, C. P., and Wood, D. M. 1978. The correlation of index properties with some basic engineering properties of soils, *Candian Geotechnica*, 15(2), 137–145.

Wyllie, D. C. 1992. *Foundations on Rock*, E & FN SPON, London.